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THE ASSOCIATION was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of 800 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

A Life Member shall be a member whose service to the Association entitles him to special recognition by the Association.

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An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

The initiation fees and annual dues are as follows:

Initiation Fees		*Annual Dues*	
Members	$ 3.00	Members	$ 6.00
Juniors	1.00	Juniors	3.00
Associates	10.00	Associates	20.00
Corporate Members	10.00	Corporate Members	10.00

This Association has six regular meetings each year, all of which, except the annual convention in September, are held at Boston.

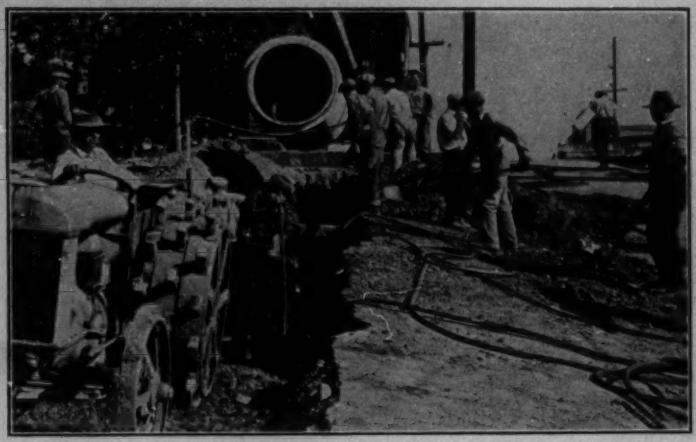

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Table of Contents

PAGE

Portrait of President Richard H. Ellis ..... *Frontispiece*

The New High-service Water Supply of Concord, N. H.
By Percy R. Sanders and Frank A. Marston 1

Experience with Ammonia-Chlorine Treatment of the East Providence (Rhode Island) Water Supply.
By James V. Turner and Charles L. Pool 13

Replacement of Brook Crossing with Bridge Crossing on 42-inch Pipe Line. By Harold W. Griswold 22

The Design of Pipe Lines. By Karl R. Kennison 27

Manganese in Impounded Water Supplies.
By Robert Spurr Weston and Attmore E. Griffin 40

Reconstruction of the Lebanon, N. H., Water Works.
By Richard S. Holmgren 52

The High-speed Diesel Engine as a Prime Mover for Pumping Stations. By Elliott Earl 61

Relation of Geology to the Groundwater Supplies of New England.
By Irving B. Crosby 74

Proceedings:
- January Meeting 96
- February Meeting 97
- Water Works School 98

Obituary. John R. Freeman 100

It was concluded that the Soucook River which lies to the east of Concord, on the opposite side of the Merrimack River from the city proper, afforded the best opportunity for increasing the available water supply to the extent of at least 1 m.g.d. The Soucook River flows in a southerly direction into the Merrimack forming the easterly boundary of the city and discharging at a point about 3 miles below Concord. Above a point east of Concord Plains where it was proposed to take the water, the watershed has an area of 82 sq. miles. The yield of such a drainage area would be sufficient with proper development to meet the water requirements of Concord for many years. The river water is comparatively free from pollution but is subject to possible contamination from scattered camps and communities the largest of which is the village of Loudon located about 6 miles above the intake site. The population on the watershed is about 25 per sq. mile.

The first plan was to build a small dam across the stream to form a pool from which the river water could be pumped to a color and iron-removal plant. During normal flows the river water has a color of 35 p.p.m. and contains iron to the extent of 0.35 p.p.m., both being sufficient to warrant treatment.

In 1930, further studies were made in the Soucook River valley to determine the possibility of finding a supply of ground water in some of the flood plains bordering the river in the hopes that a sufficient quantity of water of good character might be obtained and the cost of treatment thus avoided.

Preliminary Test Wells. The first group of test wells was located on the westerly side of the Soucook River in the town of Pembroke, seven 2½-in. wells being driven to depths varying from 26 to 47 ft. The materials encountered consisted of fine and coarse sand of a yellowish color. The wells were located about 100 ft. from the river and each yielded about 20 g.p.m. The results of individual analyses of samples of water from these wells indicated an iron content varying from 1.50 to 0.25 p.p.m. Two of the wells were shut off, and the iron content of the remaining wells became 0.6 p.p.m. The water was acceptable from a sanitary standpoint in all other respects. Since these test wells covered a wide area, it appeared that iron-free water could not be easily obtained in the general vicinity. The 5 wells were connected to a suction main and a pumping test conducted for four weeks, during which time the water was passed through an experimental iron-removal plant.

These wells were put down by a crew of 5 men, using a gasoline hoist and the usual tripod rig with a driving weight. The well pipe was cut into 6-ft. lengths.

Experimental Iron Removal Plant. The iron removal plant consisted of a series of wooden tanks so arranged as to include the processes of aëration, coke prefiltration, sedimentation and rapid sand-filtration. The plant was designed for a maximum rate of flow of 18 g.p.m., the flow being

controlled by a constant-head orifice capable of variation from 18 to 10 g.p.m. The aërator, consisting of 4 nozzles, and coke filter were combined in a single wooden box. The sedimentation basin was equipped with draw-off cocks so that a period of sedimentation of 1, 2, 3 or 4 hours could be obtained at the maximum rate of flow. The filter was constructed in a manner similar to the ordinary rapid sand-filter with perforated pipe underdrains, 18 in. of graded gravel and 30 in. of sand.

The iron-removal plant when operated at practicable rates showed excellent results in removing iron from the raw water, which contained as much as 2.0 p.p.m. at times. The effluent was always clear and colorless with a final iron content varying from 0.01 to 0.08 p.p.m.

The principal results of the tests and the average iron contents at each stage of the process were as follows:

	Iron Content (p.p.m.) at Rate of Flow of:	
	18 g. p. m.	10 g. p. m.
Raw Water	1.39	0.93
Aërated Water	1.39	0.93
Coke Filtered Water	0.74	0.24
Settled Water	0.71	0.15
Sand Filtered Water	0.08	0.01

The period of sedimentation was 4 hours and the rate of filtration was 2 g.p.m. per sq. ft. of sand area. The hardness varied from 13 to 15 p.p.m. and the alkalinity from 15 to 18 p.p.m. Color, odor and turbidity of the plant effluent were zero.

As an experiment, river water was passed through the plant for a short time, to determine the feasibility of treating that water in this manner. The results indicated that the iron was in a different state than in the ground water, probably combined with humic acids or already in an oxidized colloidal condition. The river water passing through the plant was practically unaffected by aëration, sedimentation or filtration but could be treated by using a coagulant to precipitate the color and colloidal matter. No coagulant was used with the ground water.

Additional Test Wells. While the iron-removal plant was in operation it appeared that there might be some merit in exploring the easterly bank of the Soucook River on a flood plain lying north of the test-well field. Sixteen 2½-in. wells were, therefore, driven to depths varying from 26 to 68 ft. in such manner as to outline a suitable area for a well field.

In general, ground water was encountered from 5 to 7 ft. below the ground surface and ledge was struck at a depth of about 50 ft. In all, about 40 test wells were put down. Records were kept of the materials penetrated, and samples were collected of characteristic strata. Using a gasoline engine driven diaphragm pump, pumping tests were made on each well, with the strainer at various depths, the last test being with the top of the strainer 20 ft. below ground-water level. The rates of pumping varied from 20 to 60 g.p.m. per well.

The average time for driving a well, making all tests and pulling the pipe was about 9 hours.

The strata on the east side of the river were similar to those encountered on the Concord side, the material being principally fine and coarse sand, with some slight admixture of fine gravel. The results of individual analyses of the water from these wells indicated that certain of the wells were capable of yielding a substantially iron-free water while others, restricted to small areas, showed the effect of iron-bearing deposits. These test wells indicated that a well field yielding as much as 1 m.g.d. could be laid out adjacent to the river.

Pembroke Well Field. The well field as finally laid out is shown in Figure 1. Originally 100 wells were driven and connected, but the shortage of water becoming more acute, 50 wells were added, making a total of 150. These wells are 2½ in. in size and are driven to depths varying from 30 to 50 ft. The test yield of individual wells varied from 15 to 70 g.p.m. The materials encountered in driving the wells consisted of layers of fine and coarse sand and fine gravel without boulders. The well field covers about 28 acres.

Wells and Connections. The wells were made up and were driven in accordance with usual practice. The strainers manufactured by A. D. Cook, Inc. had a diameter of 2½ in. and a length of 6 ft. with No. 30 slot openings. The type of strainer selected required that the well pipe be driven to the full depth and the strainer placed inside the opening. The pipe casing was subsequently partly wthdrawn, exposing the strainer, and a lead packer was swedged against the inside of the well casing, to prevent the entrance of sand.

The well casing consisted of 2½-in. extra heavy genuine wrought iron pipe cut off so that the capped top extended about 18 in. above the ground. (See Fig. 2.)

In driving the permanent wells, two gangs of five men each were used, and there was one extra man to assist on piping and other work. Considerable work was required to clear brush so that the driving outfits could be moved from well to well. In addition deep snow and severe cold helped to slow up the work. The average rate of progress was 1 well per crew per day. A view of one of the well-driving rigs with crew is shown in Figure 3. Fifty permanent wells were put down between June 2 and July 2, 1931.

The connections were made about 4½ ft. below the surface of the ground. A single sweep tee was inserted in the well casing and a 2½-in. heavy lead gooseneck (11 lb. per ft.) installed in the branch, followed by a short nipple, a 2½-in. flanged union and a 2½-in. iron-body bronze-mounted gate valve, equipped with valve box. Sufficient 2½-in. standard weight black wrought-iron cement-lined pipe was added to make the total length of connection approximately 10 ft. The connection at the suction main was made by inserting a tapped 4-in. plug in the 4-in. bell outlet of the tee in the suction main. The poured lead connection at the suction main,

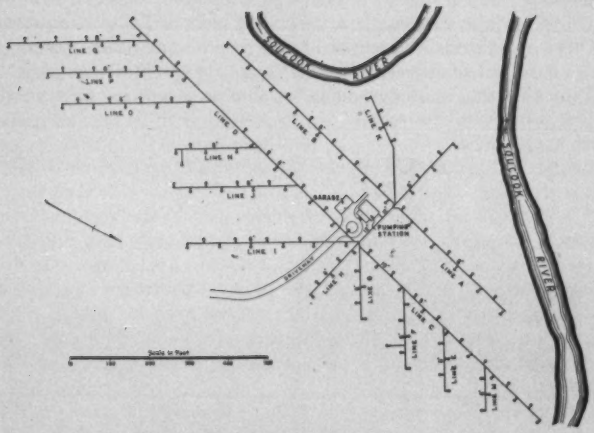

FIG. 1.— PEMBROKE WELL SYSTEM.
Plan of Well Field, Concord, N. H., Water Works.

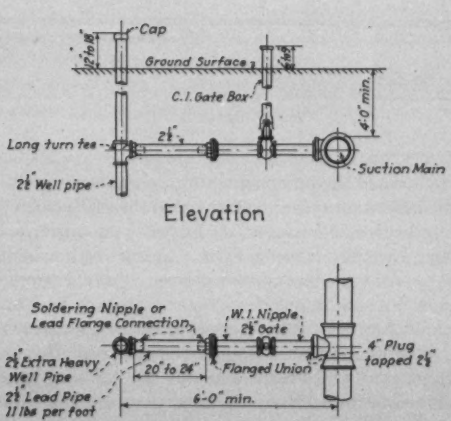

FIG. 2.—TYPICAL WELL CONNECTION.

together with the lead gooseneck, afforded sufficient flexibility for easy connection. All of the gate valves used on well connections were air tested to 100 lb. per sq. in.

The suction main consisted of 8-in., 10-in., and 12-in. cement-lined sand-cast pipe installed with bell and spigot joints filled with lead. The suction main was carefully laid to grade with an upward pitch of about

FIG. 3.— WELL-DRIVING RIG AND CREW.

3 in. in 100 ft. toward the pumping station to permit entrained air to rise to the sand chamber for removal. After laying the suction main and calking the joints, a test with compressed air, 80 lb. per sq. in. pressure, was applied, the joints being carefully investigated for leakage with a mixture of soap suds and water. All visible leaks were stopped. Careful attention was paid to this detail in order to reduce the quantity of air to be removed by the priming equipment and to reduce the cost of maintenance and operation.

At first, it was found that nearly all joints leaked. This was caused by small particles of sand which had adhered to the bell of the pipe and which were not removed in the ordinary cleaning process. In all later work, the spigots and bells of the pipe were thoroughly cleaned with cold chisels and wire brushes, loose dirt was carefully removed and in some instances, where it was hard to get an air-tight joint, a little melted tar was poured into the joint just before pouring the lead, thus making it possible to get full contact between the lead and the coating on the adjacent iron surfaces. Consider-

able trouble was had, also, from leaks in the suction mains caused by settlement of the pipe when frozen lumps of earth back-filling thawed out as the temperature was raised when ground-water flowed through the pipes. It was necessary to go over the lines and connections again to remedy this difficulty.

Force Main. A 12-in. cast-iron pipe was laid from the pumping station a distance of 15 636 ft., to connect with the high-service distribution system of the city. For some length this line serves as the main supply for the "Plains" district. The work involved a crossing of the Merrimack River about 500 ft. long. This was made by laying 12-in. Universal pipe on steel supports built on an existing highway bridge. To protect the pipe, a wooden boxing was constructed around it and the pipe was packed in coarse granulated cork.

A section of this pipe, 7 136 ft. long, was tested after it had been in use about 6 months, to determine the value of the coefficient of friction "C" in the Hazen and Williams formula. The rate of discharge of water was measured by nozzle piezometers and by the station Venturi meter. There were no services in use at the time. The drop in pressure was measured by calibrated pressure gages. Two test runs gave values of C equal to 116 and 118. The pipe tested was 12-in. cement-lined sand-spun pipe.

Emergency Pumping Equipment. During the winter of 1930–31 the water level in Penacook Lake continued to drop, and a heavy sheet of ice formed on the reservoir so reducing the available volume of stored water that there was danger of the supply becoming insufficient to meet the demand.

Work had already been under way on the new wells since November 10, 1930, and was continued during the winter together with the laying of the suction mains. By February 7, 1931, one hundred wells had been driven and connected. The pumping equipment for the station had been ordered but could not be delivered and installed in the new station in time to relieve the impending shortage of water. Accordingly, an emergency pumping station was built near the site of the new building, and in it was installed a used centrifugal pump of 750-g.p.m. capacity for a head of 100 ft. This pump had been in use for some years as a standby for fire protection. It was the only pump on the market immediately available with motor and control panel suitable for the purpose. Its purchase was a most fortunate event. Priming of the centrifugal pump was provided for by installing a Nash Hytor vacuum pump purchased from stock. A used sand chamber 54 in. in diameter and 7 ft. high was also obtained, and with the Venturi meter intended for the permanent station the equipment was completed.

Pumping was started with the emergency equipment on February 9, 1931, ninety-one days after work began on the wells. With this plant, an average of 750 000 g.p.d. were pumped into the high-service system until September 29, 1931 when the plant was superseded by the permanent station. In addition, water was purchased from the Penacook and Bos-

cawen Water Works at a rate of 150 000 g.p.d. to supply a small district in Penacook and West Concord.

In these ways, the danger of a shortage of water was avoided, and it was not necessary at any time to curtail consumers on the use of water, the only restrictions imposed being on the flushing of streets and the flooding of skating rinks.

Permanent Pumping Equipment. The equipment of the new station includes three pumping units as follows: (see Fig. 4).

Two 6-inch, 2-stage, series connected, Morris centrifugal pumps, capacity 1 150 g.p.m. each, total head 307 and 312 ft. respectively, pump

FIG. 4.— PEMBROKE PUMPING STATION.
Interior of Pump Room, Concord, N. H., Water Works.

efficiency 78 and 77 per cent. respectively, direct connected to 125-h.p. Westinghouse wound rotor induction motors, 1 800 r.p.m. synchronous speed for 3-phase, 60-cycle, 2 200 volt electric current.

One 6-inch, 3-stage, series connected, Morris centrifugal pump, capacity 1 150 g.p.m., total head 310 ft., direct connected to 175-h.p. Sterling Dolphin C-6 cylinder gasoline engine, 1 200 r.p.m.

The gasoline engine is equipped with all necessary accessories, including a muffler, cooling-water feed tank, dual ignition, storage battery and charger, fuel pump, etc. A 1 000-gal. gasoline storage tank is provided underground outside the station. The flexible couplings are Falk all-steel couplings.

Electrical Control Equipment. The switchboard, manufactured by Sundh Electric Company, Inc., consists of five panels, an incoming line panel, a meter panel, two motor feeder panels and a station service panel. The panels contain recording and indicating meters, all necessary switches

and protective devices and an electrical interlock to prevent both motor-driven pumps from being operated at the same time.

The switchboard is of the "dead-front" type, that is, the operating panels from which all routine control of the station is handled, have mounted on their faces and accessible to the operator only devices carrying the low potential, and these devices are carefully insulated to protect against accident to the operator and damage due to leakage of water from piping or pumps. The front panels are of steel and the rear panels are of ebony asbestos. The electrical equipment is installed in a special room conveniently arranged for access to all wiring and connections.

Miscellaneous Station Equipment. The station is equipped with a slate gage board on which are mounted an electric clock, indicating and recording water-pressure gages and indicating and recording vacuum gages. There is a Venturi meter register-indicator-recorder made by the Builders

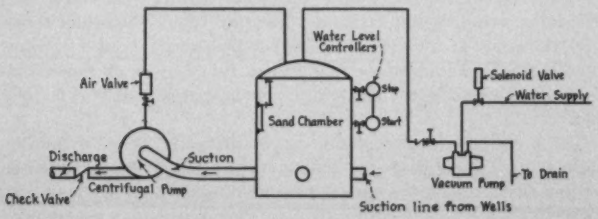

FIG. 5.— DIAGRAM OF PUMP PRIMING SYSTEM.

Iron Foundry to record the water pumped. A sewage ejector is provided in the pump room to receive the drainage from the toilet and lift it to the sewage treatment plant located some distance from the well field. An automatic sump pump provides for the removal of drainage water.

Pump Priming System. The center of the motor-driven pumps is about 6 ft. above the normal ground-water level and about 10 ft. above the average water level as drawn down during pumping. To prime these pumps and to keep them primed as well as to remove air from the suction piping and vacuum chamber or sand catcher, an automatic priming system was installed. This consists essentially of:

Two Nash Hytor, vacuum pumps, size AL5, bronze fitted, capacity 8½ cu. ft. of air per min. at 20-in. vacuum each, speed 1 150 r.p.m. direct-connected to 2-h.p. General Electric induction motors, 3-phase, 60 cycle, 220 volt current. One of these units has an auxiliary drive furnished by a 4-h.p. Novo, size T.U. gasoline engine for use in case of power failure.

One steel vacuum or sand chamber, 6-ft. diam., by 12 ft. high equipped with two sets of 24-in. gage glasses and four float-operated Time-O-Stat switches arranged to start and stop the vacuum pumps on the fall and rise of the water level in the vacuum chamber.

The general arrangement of the various units and piping is shown diagrammatically in Figure 5. In the priming pipe above each pumping unit is a Crispin air and vacuum valve designed to prevent the flow of water into the priming piping when the centrifugal pump has been primed and begins to discharge. The priming pipe is connected not only to the top of the pump casing as shown, but also to the suction lobe of the casing.

Although the total length of suction piping in the well field is 6 000 ft. and the length of well casings and connections is 7 800 ft., the system is so tight that one priming pump operates on the average only once in 80 minutes and then for a period of but 13 minutes.

The pumping equipment, priming system and electrical control apparatus were furnished and installed under contract with Starkweather and Broadhurst, Inc. of Boston.

Pumping Station Building and Garage. The pumping station is 46 ft. by 42 ft. overall in plan and the garage 45 ft. by 24 ft. The pumping station consists of a main pump room two stories high, an office, hallway, locker room, toilet, heater room and electrical bay. The heater room and electrical bay are at the same floor level as the pumps about 6 ft. below the ground surface. Access to the pump room is had by an entrance from the hallway on the first floor. The pump room is approximately 44 ft. by 28 ft. inside.

The building is constructed of brick with concrete sub-structure and granite trim. The bricks are waterstruck, common bricks, selected for color and texture. At the rear of the station, provision has been made in the concrete work for a future extension, should it become necessary. The interior of the pump room is lined with dull glazed yellow vitreous tile to a height of about 10 ft. above the pump-room floor, the remainder of the exposed building wall being of a light buff brick. The floor is covered with dark red quarry tiles, with darker tile used at the edges, to form borders. The interiors of the office and toilet are finished with buff brick. Metal sash is used throughout, and all doors and frames are of hollow metal construction.

The pumping station is equipped with a complete toilet including lavatory, water closet, slop sink and drinking fountain. The heating system is of the low-pressure vacuum type. Steam is furnished by a cast iron boiler fired by a Gilbert & Barker oil burner. Unit industrial heaters are installed in the pump room and shop while the smaller rooms are equipped with cast-iron radiators. The station and garage are lighted by electrical fixtures selected to give well diffused illumination. Exterior lights for the driveway and front yard are provided at the front door of the pumping station and along the face of the garage.

The garage provides space for three cars or trucks, together with a small workshop. This building is of construction similar to that of the pumping station. The garage doors are of the overhead disappearing type of Barber-Coleman manufacture. The garage and shop are kept warm by

unit heaters supplied by steam from the pumping station. The pumping station building and garage were built under contract with the Norquist Construction Company of Concord, New Hampshire. Plans and specifications for the pumping station, garage and equipment were prepared and construction was supervised by Metcalf & Eddy in coöperation with the Superintendent of the Water Department.

Sewage Disposal Plant. The sewerage system consists of the sewage ejector in the pumping station, a force main about 640 ft. long and two leaching cesspools with 100 ft. of filter trenches. The plant is located downstream from the well field so as to avoid contamination of the water supply.

Construction Costs. The pumping station was built during a time when advantage could be taken of favorable construction costs. Pipe laying, well driving and road building were carried out by water department forces under the direction of the Superintendent. A summary of the principal items together with their cost is given in the following table:

Cost of Soucook Water Supply System.

Test Wells	$4 725
Force Main	17 910
Permanent Wells and Suction Piping	44 573
Pumping Station and Garage	
Buildings	34 209
Equipment	18 302
Grading, drainage, sewage disposal and miscellaneous	5 050
Roads	3 343
Land	10 333
Engineering and superintendence	23 705
Miscellaneous	428
	$162 578

Operation of Station. The electric motor-driven pumps are operated alternately, one week at a time for about 17 hours each day. The engine-driven pump is run for about 1 hour each week. The average rate of pumping is 1.13 m.g.d., operating for 16.75 hours. The largest amount pumped in one day was on July 30, 1932 when 1.565 m.g. were pumped in 24 hours. The station is run by a single attendant. There are 3 shifts of 8 hours each, 7 days per week. A relief man is available so that each operator works but 6 days per week.

All of the water pumped from this station is used in the high-service system and amounts to about 40 per cent. of the total consumption of the city, the balance being supplied by gravity from Penacook Lake. The normal ground-water level in the well field (no pumping) stands at about the same level as that of the river. During the dry period on July 31, 1932, the ground-water level dropped a maximum, in one of 6 observation wells,

of 6.7 ft. below river level, whereas on November 24 after a heavy rain the ground water at the same place was only 4.2 ft. below river level. After the pumps have been stopped for 7 hours the ground water level rises about 0.35 ft. The lowest water level maintained in the vacuum chamber is 3 ft. above the center of the pumps and the highest level is 5.9 ft. above the center. The average electric-power consumption for pumping water, operating vacuum pumps, sewage ejector and sump pump amounts to 1 465 k.w.h. per million gallons of water pumped to the distribution system. This is equivalent to 4.8 k.w.h. per million foot gallons. The static pressure on the pump discharge is 90 lb. per sq. in.; the dynamic pressure is 125 lb. per sq. in. The vacuum when the pumps are running is equivalent to 15 ft. of water; when shut down, it drops in a very few minutes to 10 ft.

The work has been carried out under the direction of the Board of Water Commissioners of which Mr. Harry H. Dudley is President. Credit is also due to members of the staff of Metcalf & Eddy, Messrs. H. E. Bailey, Assistant Engineer, C. E. Greene, Mechanical Engineer, and W. R. Bradlee, Resident Engineer. The architectural design was prepared by Messrs. Frank A. and Frederic M. Kendall of Boston.

EXPERIENCE WITH AMMONIA-CHLORINE TREATMENT OF THE EAST PROVIDENCE (RHODE ISLAND) WATER SUPPLY.

BY JAMES V. TURNER* AND CHARLES L. POOL.†

[*Read September 28, 1932.*]

This paper gives a brief general description of the East Providence water supply to which ammonia-chlorine treatment has been applied continuously for the past 18 months. In this connection it is thought better to present a fairly comprehensive picture of a supply to which this treatment has been successfully applied than to confine the discussion to facts strictly relevant to the ammonia-chlorine process.

First, we wish to express appreciation of the technical operation of the plant and the keeping of records by Mr. G. L. Wilkinson, Chief Engineer, and of an intensive study of two months' duration by Mr. W. B. Robinson, Asst. Chemist of the Rhode Island Public Health Commission.

Description of Supply.

The town of East Providence is located on the easterly side of the Seekonk and Providence rivers, is bounded on the north by the city of Pawtucket, R. I., and on the east by the town of Seekonk, Massachusetts. Its population of 30 000 makes it the largest town in the United States with one exception. About a square mile of the area in the closely built up section, called the Watchemoket Fire District, has from the outset received its water supply from the city of Pawtucket. The rest of the area of some twelve square miles is supplied with water by the East Providence Water Department. It is estimated that the population served is 20 000.

The Watchemoket Fire District, a quasi-municipal corporation incorporated by an act of the General Assembly on June 13, 1878, serves the compact part of the town, known as Watchemoket Village. The water is purchased from the city of Pawtucket and delivered to the district-owned distribution system through a 12-in. main laid by the district to the East Providence-Pawtucket line on Pawtucket Avenue. Water and fire service for the Watchemoket Fire District have been available since about 1882.

The East Providence Fire District, comprising all that portion of the town of East Providence not included within the limits of the Watchemoket Fire District, was incorporated as a quasi-municipal corporation by the General Assembly in May, 1891. Construction of the plant proceeded during the years 1892–1894; water became available in 1892 or 1893, and construction was completed in the fall of 1893.

*Superintendent of Water Supply, East Providence, R. I.
†Sanitary Engineer, R. I. Public Health Commission, Providence, R. I.

The East Providence Water Company was incorporated by an act of the General Assembly on May 22, 1895, and by its charter was granted the power to purchase and acquire from the East Providence Fire District its existing water-works plant and other property, and the East Providence Fire District was granted the power to sell its plant to the company. Acting under this authority, the East Providence Fire District, on October 8, 1895 sold, and the company purchased, the existing water-works plant and other property of the District. The plant was subsequently operated by the East Providence Water Company until 1928 when the town purchased the supply from the company. Since then the town has operated the plant under the Board of Water Commissioners.

For many years after construction of the works, pumping was done by a water-power plant supplemented by a steam-driven pump. The steam equipment was later discarded for electrically-driven centrifugal pumps, and in 1924, the water-power plant having depreciated to such an extent as to make its further use inadvisable, a new wheel of 230-h.p. capacity, driving a generator of approximately the same capacity, was substituted. This is used whenever the flow of the river is sufficient to generate power to drive the pumps. The department derives its main supply of electricity by purchase of power under special contract from the Narragansett Electric Light Company.

At the pumping station is a group of structures, consisting of a stone building, a wooden filter house with masonry foundations, and an outdoor uncovered concrete sedimentation basin which receives water through a steel penstock from Ten Mile River. The treated water is delivered by centrifugal pumps driven by electric motors. There are four units, three on the high-service and one on the low-service system. The high-service system comprises the domestic, fire protection, commercial, and industrial services. The low-service system is confined to the supply of the Sayles Finishing Plants, Inc. and is not now in use. The water pumped to the domestic and commercial consumers, including the municipal services, is pumped against a head of about 300 ft. The low-service pump has a capacity of about 1.5 m.g.d. and is driven by a 50-h.p. electric motor. The high-service pumps comprise three units, with capacities of about 1.5, 1.75 and 1 m.g.d. The two larger and older units of the high-pressure service have motors of 200-h.p. capacity.

The water from the Ten Mile River which has to be treated comes from a watershed of about 54 square miles, of which 48 are in the Massachusetts towns of Seekonk, Attleboro, Mansfield, Wrentham, and North Attleboro. The remaining six square miles are in Rhode Island.

While a considerable portion of the watershed is sparsely settled, much of the area immediately adjacent to streams is thickly populated, including the centers of Attleboro and North Attleboro, in the respective towns of about 22 thousand and 10 thousand people. These centers have sewerage systems discharging an average of about 1 m.g.d. and ⅔ m.g.d. respectively.

In each case the sewage is filtered at an average rate of some 75 000 g.p.d. per acre. In addition, several jewelry factories in North Attleboro, half a dozen mills in Attleboro, one of which discharges about ¾ m.g.d. of industrial sewage, another half dozen smaller mills below Attleboro and various small communities discharge wastes into the streams, most of which are trade wastes. These include sewage in some cases. Inquiry into the character of industrial wastes failed to reveal the presence of substances likely to contribute phenolic taste troubles experienced in the past. Much of the immediate watershed contains swampy areas.

Some of the unfavorable conditions are mitigated by ponds which provide storage, but the raw water arriving at the plant shows the effects of pollution. The water fluctuates widely in quality. In the past 18 months, for example, the color has ranged from 20 to 120 p.p.m.

The water supply system consists first of a storage reservoir, known as Old Central Reservoir, with a capacity of approximately 50 m.g. The average daily run-off is slightly in excess of this volume, but varies during different seasons of the year. Under extreme low-water conditions the run-off averages from 12 to 20 m.g.d. During the drought of 1923, the minimum run-off was about 12 m.g.d. It seems probable that over a long period of years, such as a century, the minimum run-off would not be estimated at less than 8 m.g.d., and it would perhaps be as much as 10 or 12 m.g.d. As the present consumption is about 2 m.g.d., it seems evident without closer analysis that there is ample supply for present and future needs.

From the storage reservoir the water flows through the Ten Mile River to the point of diversion at the pumping station at the so-called Hunts Mills Dam, built on a rocky barrier across the stream. Here water is diverted, through the head gates, the wheel flume and a penstock, into the pumping station where a portion of it is used for generating electric power to drive the pumps. Another portion is treated prior to consumption by the district supplied by the department.

The treatment consists briefly of about three hours retention in a settling basin after addition of alum as a flocculating agent; subsequent treatment with ammonia, soda ash, and chlorine; passage through rapid sand filters and storage in a clear well whence it is pumped into the system.

Alum is added at a rate of 2 to 2.5 grains per gallon from a dry-feed machine at a point about 135 ft. ahead of the settling basin, outside of the filter house. The settling basin, holding about 400 000 gal. and built of reinforced concrete, is of good design and enduring type. The basin is emptied and cleaned regularly every three months. This operation requires about six hours, two hours to empty, one hour to flush out with a fire hose and three hours to refill. At this time prechlorination is followed by chlorination of the effluent from the filters.

At the end of the settling basin and at the mouth of the 24-in. take-off pipe, ammonia is added at a rate of 0.24 p.p.m. in summer and 0.14 p.p.m. in winter. Before chlorine is added, soda ash is applied from a dry feed

machine at a rate of 0.4 grain per gallon to adjust the pH. At the point where the feed pipe enters the filter house it is reduced from 24 to 16 in. The distance from the point of application of ammonia to that of soda ash is about 70 ft. Five feet beyond this point chlorine is added. The usual chlorine dosages have been 0.75 p.p.m. in summer and 0.42 p.p.m. in winter.

After the chlorine has been put in, the water passes directly to the six filters. The distance from the point of application of chlorine to the first two filters is 13 ft., to the next two 40 ft. and the last two 55 ft. The distance from the main line to each filter is seven feet, the branch pipes being 8-in. in size.

The filters are 15-ft. circular Jewell gravity filters with Weston automatic control. They contain a 14-in. layer of gravel in graded sizes, on top of which is a 30-in. layer of special filter sand.* The rate of flow through the filters is automatically controlled at 2 g.p.m. per square foot or about 350 g.p.m. per filter. The filters are given a regular 10-minute wash two or three times per 24 hours, depending on conditions. The rate of flow of wash water is 1 300 g.p.m. per filter or 12 in. per minute vertical rise. The amount of wash water used is about 3½ per cent. of the total amount filtered. During the wash, water is forced up through the bottom while the surface of the sand is slightly agitated with revolving rakes. The filters may be given a semi-wash or "blow" for several minutes whenever the loss of head is not sufficient to warrant a complete wash.

After passing through the filters, the water goes to the clear well which has a capacity of 196 000 gallons and occupies the floor beneath the filter room. From the well, the water is pumped into the distribution system and standpipe. The pressure at the pumping station is 135–150 lb. per sq. inch. The standpipe has a capacity of 1 000 000 gal., is located about one mile from the plant and assists in caring for sections more distant from the plant. The farthest user is about five miles from the pumping station, the nearest is about 400 ft.

Chloramine Treatment.

Chloramine treatment was started in March, 1931. Although it was felt that a drinking water of very high quality was being delivered, considering the raw material, improvement was desired. Under certain conditions, consumers would complain from time to time about unpleasant tastes and odors which were variously described as phenolic, medicinal, chlorinous, etc. Previous to March, 1931 the filter effluent was chlorinated. Upon careful scrutiny of the then quite new chloramine treatment, it was decided to try it out on a plant scale. The results obtained have given satisfaction.

Tastes and odors are now successfully controlled during occasional appearances of compounds which used to cause the water to taste badly

*An effective size of 0.59 mm. and a uniformity coefficient of 1.39 was found for one filter studied by Joseph Wuraftic, Asst. Sanitary Engineer, R. I. Public Health Commission during a study (not published) of filter-washing in which it was found that East Providence showed more efficient washing than another more modern plant in Rhode Island and a modern Massachusetts plant.

when treated with chlorine alone. It is now possible to carry a much higher residual chlorine without taste. A decided improvement in the tendency towards aftergrowths in the clear well has resulted. Before chloramine treatment the pipes in the clear well would be encrusted in places with hard strong growths which were knocked off with difficulty with a hammer. Growths have been observed during the new treatment, but they are of a soft nature and can be brushed off easily with the hand. Also, the filters remain much sweeter and cleaner, even though records show no indications of increased lengths of run or of a decrease in the proportion of wash water used to total water supplied. In general it can be said that a much more uniform water is obtained. Customers' complaints have been reduced to practically zero, whereas formerly their number was formidable.

Before starting the new treatment, the effluent from the filters was good bacteriologically, the subsequent continuous chlorination providing an added safeguard. The filter effluents only occasionally showed the presence of *Bact. coli*, and it was not at all uncommon to obtain a zero total count at 37° C., while the total number of bacteria was normally very low in the filtered water. In not one single sample had the final effluent as delivered showed the presence of *Bact. coli* in a 10-cc. portion since 1928, according to analyses made twice a month by the Rhode Island Public Health Commission. The raw water rarely fails to show *Bact. coli*, frequently revealing 100 per cc. while the total count at 37° C., often runs into the thousands.

A laboratory is maintained at the plant where all the tests essential to operation are made daily, exclusive of bacteriological analyses. The orthotolidin test is relied on for daily assurance of safety.

It will be readily appreciated from the recital of water quality and the description of the watershed that in the interest of safety few supplies in New England need to be treated more carefully than this one. Consequently, when the change from chlorination of the filter effluent was made to ammonia-chlorine treatment of the influent, it was necessary to make certain that the routine operation was adequate and reliable. For a time, until satisfied of this, chlorination of the filter effluent was continued in addition to pre-disinfection.

Three questions had to be answered at the outset, as follows:

(1) Was the fact that the ammonia-chlorine process is slower in action than chlorine alone quantitatively serious in this case where consumption of the water begins 400 ft. from the pumping station?

(2) Could the bacteriological analyses made in the state laboratory about 1½ hours after collection of samples be relied on to depict conditions that actually existed at the time of sampling at the plant or at consumers' faucets within perhaps less than 1½ hours of travel of the water between chlorination and consumption? In other words, in view of the new importance of the time element, would the extra time of contact with chlorine in the sample bottles during transit give misleading results for the samples examined?

(3) Could the ortho-tolidin tests be relied on as before to predict that the residual chlorine values normally carried would still ensure adequate sterilization?

It was estimated that an average of six minutes elapsed between the clear well and the nearest consumer. The detention period in the well which is normally part full averages perhaps an hour, but probably water at times passes out of it almost without noticeable detention. An average time of 13 minutes for passage through the filters was estimated, and from the point of chlorination to the filters 1 minute was allowed. Therefore, twenty minutes of disinfecting action was all that could be depended on before the first possible consumption, though the average time for the nearest group of consumers would run from 1½ to 2 hours.

Special sets of samples were examined bacteriologically to compare results obtained on incubating immediately after sampling, with results obtained upon incubating after the normal lapse of 1½ hours. The averages on these sets of samples for ammoniated and chlorinated filter influents and effluents did not show any appreciable differences, the bacterial results under the operating conditions being so low as to be considered uniformly excellent.

While this procedure answered both questions (1) and (2) in a fairly assuring manner, the necessity of careful analytical control by the ortho-tolidin test and bacteriological analyses has been increased. A typical set of these comparative analyses for April 30, 1931 follows:

Time of Sampling.	Time of Incubation.	Source.	Total Bacteria.		Red Colonies Litmus Lactose Agar 37° C.	*Bact. coli* per 10cc.	Residual Chlorine p.p.m.
			20° C.	37° C.			
3:15	4:30	Raw water	700	125	4	100	....
3:15	4:30	Settled water	220	100	2	10	....
3:15	3:15	Influent to filter	18	14	0	0	0.40
3:15	4:30	No. 4	12	25	0	1	0.40
3:15	3:15	Effluent from filter	0	0	0	0	0.12
3:15	4:30	No. 4	0	0	0	0	0.12
3:15	3:15	Effluent from filter	0	0	0	0	0.07
3:15	4:30	No. 2	0	1	0	0	0.07
3:15	3:15	Final effluent from clear	0	0	0	0	0.10
3:15	4:30	well	0	1	0	0	0.10
3:15	4:30	Tap sample	20	17	0	0	0.00

Since it has not been practicable to continue the bacteriological checks at the filter plant, a routine has been subscquently established of adding enough sterilized sodium thiosulphate solution to duplicate sets of sample bottles to eliminate further disinfecting action by the chlorine between the time of sampling and incubation. Results on these samples show that there are times when the average of 1½ hours' contact in the sample bottle needs to be eliminated. With high chlorine residuals discrepancies appear in the 20° C. counts for the filter influents and tend to appear in the 37° C. incubations. The effluents with lower residuals show the same tendencies in a lesser degree, but the final mixed effluent has not shown significant differences.

The answer to question (3) whether the ortho-tolidin tests could be relied on to the same extent as though chlorine alone were used has had to await the accumulation of data as bacteriological findings were gradually correlated against the residual chlorine readings at the plant. Higher residuals than formerly must be carried.

The bacteriological results have been entirely satisfactory since the inception of the new treatment, not one single sample of treated water as delivered having shown the presence of *Bact. coli* in a 10-cc. portion. Total counts two or three times have shown a tendency to increase somewhat above the desired value, but a slight increase in the chlorination rate has quickly reduced them again. The water from the various filters after mixing and storing for an operating period in the clear well has never shown the presence of *Bact. coli*. Of the samples of effluents from individual filters 3.7 per cent. have shown *Bact. coli* in 10-cc. portions. Thus it is evident that while there is a margin of safety, there is no great margin to ensure perfect results for the *Bact. coli* test. Two improvements are expected to increase this margin.

The three pairs of filters, as previously noted, are each fed with water at a different distance from the point of application of chlorine. These distances are very close and insufficient mixing of the applied water results in the water actually being chlorinated at different rates from that intended to apply uniformly. Simultaneous readings will often show the free chlorine content to vary from 0.6 to 0.1 p.p.m. due to this cause. This is a physical imperfection in the layout, the significance of which was not appreciated until quite recently. It is to be corrected. The other improvement expected is shrewder control of dosages of ammonia and chlorine as a result of greater experience with the process and correlation of bacteriological results against ortho-tolidin readings. Values for residual chlorine evidently have to be interpreted differently and more cautiously than when chlorine is used alone, because of the necessity of taking into account the factors of temperature, pH, time, physical and organic characteristics, including free ammonia, etc. Variations in these factors can make a practical difference in sterilizations obtained under two different sets of conditions even though the ortho-tolidin readings may be the same for each. In addition to the

determination of the quantity of residual chlorine present, its persistence has been of interest because there appears to be a tendency for the more slowly disappearing residuals to be accompanied by slower disinfection rates.

We have wanted to outline a simple and definite procedure for the operators to follow in making free-chlorine readings by specifying time intervals between sampling and adding the reagent, and between adding the reagent and reading. In general it has been satisfactory in operating practice at this plant to add the reagent within 15 minutes of the time of sampling, according to the time of contact it is desired to simulate, and to make readings ten minutes after adding the reagent. Sometimes, however, this has given an erroneous reading. One sample last December did not develop full color until 35 minutes had elapsed. The full color has usually persisted for more than ten minutes after adding the reagent, but here again is another uncertainty. The significance of all this is obvious: that while the ortho-tolidin test is very simple to make, and is invaluable in giving indications of conditions a day or two ahead of the bacteriological results, it must be judged very cannily or else all the factors involved must be accounted for.

The aim has been to maintain a positive residual chlorine in all filter effluents. This has meant that higher residuals have been carried for filter influents than were formerly employed at any point. Residuals have not been maintained consistently in the water pumped from the clear well, and to carry a residual into the distribution system has not been attempted.

It so happened that the ratio of ammonia to chlorine of 1:3 that was initiated has been satisfactory in allowing the speed of disinfection needed on the one hand and in satisfying consumers' palates on the other. Consequently it has not been altered. The chlorine dosages have been varied between the limits of 0.40 and 0.75 p.p.m. already noted, the attempt being made to keep well above the requirements for sterilization and below the region where chlorinous tastes begin. This does not always give as wide a latitude as might be supposed. Recently an unmistakable taste, chlorinous or of some chlorine compound was obtained in the filter effluent when the residual was but 0.1 p.p.m. The taste, moreover, was sufficient to give rise to complaints. At other times a higher residual gives no taste. It is to be noted that the water treated is kept at pH 6.8 to 7.0, seldom exceeding these limits. Higher residuals might be carried if the proportion of ammonia to chlorine were increased, or perhaps the pH, but in both cases the disinfection rate would be slowed up.

There is a gradual tendency to maintain a higher residual chlorine content. A chlorinator of greater capacity has recently been added for that purpose. While the range is definitely restricted our needs have been met without coming too close to either border. The treatment is relatively new, and we have been reluctant to expose it to possible discredit by experiment-

ing on a plant scale with too wide a variety of dosages and ratios of ammonia to chlorine while those in use were satisfying all the requirements.

Conclusion.

At the outset of this investigation we were much helped by a paper by Gerstein* and discussion by Enslow* of an extensive laboratory study of "The Bactericidal Efficiency of the Ammonia-Chlorine Treatment." We had hoped to obtain plant data that would apply to waters outside the range of that study, and were able to prepare slides from charts suitable for verbal presentation of summarized detail figures. Upon attempt to condense the material to generalities useful for publication it has been found impracticable to do so, however, without danger of misleading our readers because of fluctuations in plant operation over a relatively short period of investigation, among which the impossibility of maintaining a uniform chlorination rate in the water feeding the several filters was most upsetting. We would be glad, however, to communicate with any officials who are considering the treatment in case our experience might be of help to them. The immediately usable information available in the ortho-tolidin test has seemed to make it desirable to attempt to correlate the factors that go along with it to affect the prediction of what sterilization will follow. A particularly interesting paper by Schmelkes† seems to offer the hope that a single measure can be developed to do what the ortho-tolidin test will not do, namely give a single value that is directly comparable with the degree of sterilization expected to ensue. This depends upon measurement of the oxidation potential of the solution containing the chlorine compound and is done electrometrically at present and expressed in units of voltage.

*Jour. Amer. Water Works Assn., 56 pp. 1334–56 (1931).
†Paper delivered at annual meeting of Amer. Public Health Assn. Oct. 24, 1932.

REPLACEMENT OF BROOK CROSSING WITH BRIDGE CROSSING ON 42-INCH PIPE LINE.

BY HAROLD W. GRISWOLD.*

[*Read January 10, 1933.*]

Hartford and the metropolitan area to the north, south and west of the city,— a territory about 11 miles long and 5 miles wide — is served with water through five principal supply mains: a 42-in. line on the north side, a 30-in. and two 20-in. lines coming into the central section, and a 42-in. line, which will eventually serve the southern section, but which at the present time has been built only to the western limit of the city.

Water is supplied by gravity, and with the exception of the southern main, which is of concrete, all of the supply mains are of cast iron with bell and spigot lead joints. The northerly 42-in. main was laid in 1917, and at a brook crossing about one-third of the way from the filter plant, which is about 5½ miles west of the city, the line had been offset north — or upstream — around a bridge over a stream about 40 ft. wide by the use of eight ⅛-bends, heavily laced and anchored by large concrete blocks and with sufficient straight pipe between the bends to produce the horizontal and vertical offsets necessary to carry the line clear of the bridge abutments and well below the stream bed.

The bridge is of concrete with three arches, the center arch larger than the other two. It was built with about two feet of fill over the top of the center arch. The piers are on a skew with the roadway and there were angle retaining walls to hold the roadway fill both upstream and downstream. The whole structure, however, did not occupy the full width of the highway. In 1917 there was plenty of room to carry the pipe line around the bridge without going outside of the highway limits. In the spring of 1932, due to greatly increased traffic, the town of West Hartford decided to install improved pavement on the roadway and to extend the bridge to the full width of the highway by carrying the three-arch section of the existing bridge both upstream and downstream.

When this construction was completed, the section of the 42-in. main crossing the brook was situated under the piers and abutments of the bridge. Even though the town had installed relieving arches over the pipe both at the piers and abutments, it was felt that a break in the pipe at this point would be so disastrous both to the bridge and to the water supply of the District that immediate steps should be taken to relocate this portion of the line and abandon the section of the pipe under the bridge.

Two relocations were possible:

*Deputy Manager and Deputy Chief Engineer, Water Bureau, Metropolitan District Commission, Hartford, Conn.

1. The pipe line could be rebuilt similar to the original construction but farther north of the bridge and outside the highway limits.

2. The pipe line could be carried over the bridge and under the north sidewalk, but only by placing headers in the 42-in. line on either side of the bridge and connecting them by several parallel lines of pipe of diameter small enough to allow them to be installed between the top of the arch and the underside of the sidewalk slab. The second plan of relocation was decided upon as being open to fewer objections than the other.

Plans were prepared for a steel header at each end of the bridge connected by three lines of 21-in. steel flanged pipe, each connecting line to

STEEL HEADER AND LINES OF 21-IN. STEEL FLANGED PIPE FOR BRIDGE CROSSING, HARTFORD, CONN.

consist of three units — a cambered center section of five in-and-out courses between flanges, with air valve connections at the top of the camber, and two straight sections to connect the cambered section to the header outlets. (See accompanying figure.) The header sections were essentially reducing fittings laid at an angle with the center line of the main line, having a 42-in. flange on the large end, a 24-in. by 21-in. reducing elbow on the small end, and two 21-in. outlets in the body of the fitting.

The material used for the pipe was flange steel — $\frac{1}{2}$-in. thick for the 21-in. pipes and $\frac{3}{4}$-in. thick for the headers. All seams were welded, and the flanges were of forged steel with corrugated copper gaskets. The pipes were tested to 300 lb. per sq. in. pressure after assembly at the shop, and

were double dipped to the Barrett Specifications. The fabrication and installation of the steel pipe and headers was done by Walsh's Steam Boiler Works of Holyoke, Mass.

As soon as the steel pipes were ready for installation, the section of the supply main in which the brook crossing was located was shut down for several days, and the drop in pressure caused by the shut down in various parts of the District was noted. It was found that pressures under ordinary drafts were only off from 6 to 10 lb. per sq. in., but since this is the largest supply main into the center of the District, good judgment dictated that the final closure be made in the minimum of time and that the preliminary portions of the work be so arranged that the main could be put into service again by simply opening the gates at each end of the section in case of extraordinary demands for water, such as might obtain during a general conflagration.

The importance of this main to the District supply was demonstrated unpleasantly on December 19, 1932, when a break in the line, about ½ half-mile toward the city from this bridge left the higher sections of the District without water for about an hour and a half and dropped the pressure in the business section about 50 lb. per sq. in.

The preliminary operations consisted in the installation of the steel lines over the bridge with the headers at each end, the uncovering of the cast-iron line at each end of the bridge, the removal of the heavy concrete anchorages at the upper set of ⅛ bends, and the replacement of these anchorages with blocking, so that the line could be used in case of emergency. The concrete was removed with paving breakers and by the use of plugs and feathers; this was a tedious job, due to the quantity—some 50 cu. yd. — that had to be excavated.

Following the installation of the steel pipe and headers and the removal of the concrete, the connecting cast-iron piping up to points of closure, and some of the new anchorages were installed. A five-ton crane was employed for handling the pipe and fittings, and ready-mixed concrete was used for the anchorages. Both proved to be time and labor savers.

In planning for the final closure, it was suggested by J. J. Dennehy, the foreman in charge of the preliminary work, that the anchorage on each side of the bridge, below the top ⅛ bend, be left in place, that drill holes be put down through it to the level of the center of the pipe and that when the time came for final closure, the pipe below the bend, which was to be abandoned, be broken out by setting off a charge of dynamite in the drill holes.

The final plan for the closure embodied this suggestion and the work was scheduled as follows:

11:00 P. M.— Three men start to drain section.
12:00 midnight — Three men start to burn out westerly joints with oxy-acetylene torches.

6:00 A. M.— (daylight) Part of main pipe crew reports on job. Load holes in west anchorage and blast.
6:30 A. M.— Crane arrives and starts removing broken material from west side.
7:00 A. M.— Load holes in east anchorage and blast.
8:00 A. M.— Balance of main pipe crew reports on job and begins closures.
8:00 P. M.— Closures to be completed.

As in the case of most jobs, the work was not carried out exactly as scheduled, but the sequence of operations was followed. It took much longer than estimated to burn out the joints, so that the first blast was not set off until 7:00 A.M., and the second at 11:00 A.M. Both gave very successful results. The holes in the west anchorage were loaded with 12 sticks of dynamite (60 per cent. gelatin L.F.); a small mat was placed on top, then timbers were laid on the mat, and a second mat was placed on the timbers. The explosion broke the pipe off completely, cracked the severed bell end into several pieces, and shattered the anchorage so much that only two or three pieces had to be broken again in order to make them small enough to handle. The blast at the east anchorage shattered the pipe and anchorage even more than at the west. On neither side was any damage done to the adjacent $\frac{1}{8}$ bend.

As soon as the broken pipe and concrete had been cleared from the trench, the pieces adjacent to the sleeves were swung into place, and, on the west side, the closure piece was measured, cut and set. On the east side, because of the later start and because a gate was to be set in the line, progress was not so rapid, but by 5:00 P.M. it was evident that the job could be completed that night. An electric service had been installed and lights strung, with the idea of continuing the work. It was found, however, that on the east side the combination of a slight rotation of the flanged bend with a deviation from alignment in the original construction would require a closure leaving several joints wider open than good practice would approve. It was decided then to dig up an additional length of pipe on the east side, realign it and complete the closure the next day. This was done.

As soon as closure was complete, the line was filled and tested under working pressure. Leaks showed in several of the flanged joints of the steel pipe, due to their not having been pulled up hard enough when they were first made up. These leaks were remedied and the testing continued until the line showed tight under a pressure 25 lb. per sq. in. above normal. The difficulty experienced with the flanged joints on this job suggests that Dresser couplings might be used in place of flanges. After a satisfactory test had been obtained, anchorages and lacings were completed and the trench backfilled with sand.

As a precaution against abrasion of the shell of the 21-in. steel pipes with sand, due to a leak in the next pipe, the flanged joints in these pipes were packed with clay and planks driven opposite the flanges.

The total cost of the relocation was $12 000, subdivided as follows:

Steel pipe line	$3 395
Other material	4 725
Labor and equipment	3 880

This work is of interest because of the use of dynamite for breaking out a section of 42-in. pipe in preparation for changes in alignment, and because of the peculiar use of a combination of steel and cast-iron pipe to overcome difficulties of location.

THE DESIGN OF PIPE LINES.

BY KARL R. KENNISON.*

[*Received January 9, 1933.*]

This paper presents briefly certain fundamental principles and considerations that should govern the design of a pipe line, particularly with respect to the relation between the hydraulic gradient and the grade at which the pipe is actually laid, and presents the results of observations on the operation of a number of pipe lines of unusual design that illustrate some of these principles.

The particular points which are discussed are:

1. The determination of invert grades to keep air pockets at a minimum.
2. The prevention of air binding when starting the flow.
3. The automatic creation and maintenance of a vacuum to allow the safe economical location of a pipe line above the hydraulic gradient.

INTRODUCTION.

The author has had occasion to design pipe lines of various sizes and under various conditions of pressure and has been impressed with the importance of attention to the details of the invert grade. In the case of a pipe line that must be laid with high points in the line, so high that the use of ordinary air relief valves would not be satisfactory, he has been impressed with the applicability of certain fundamental principles which, if carefully observed, will allow such a pipe line to be located at a higher grade than would be dictated by the apparent ordinary practice. Such principles are fairly simple and, except for the uncertainty as to just how high a velocity the water must have in a pipe of any given diameter to cause the air to become mixed and carried along with it, it is often found to be entirely practicable to vary the depths of cuts and fills so as to give a satisfactory and economical location even though at a considerable distance above the hydraulic gradient. The water inside a closed pipe has no direct connection with the atmospheric pressure outside and can be made to do some really startling things in the way of the automatic creation of a vacuum.

The first opportunity that the author had to construct a line under conditions that would allow taking advantage of some of these principles was in the construction of a 30-in. force main about 4 000 ft. long for the city of Peabody, Mass., in 1924, by which water can be pumped into the city's storage reservoir, Suntaug Lake, at a rate of about 13 m.g.d. The shore of the lake at the point in question rises to an elevation about 20 ft. above the lake level, so that it was necessary either to pump against an

*Designing Engineer, Metropolitan District Water Supply Commission, Boston, Mass.

extra head of 10 or 15 ft. to get over the top of this hill or to pay for very deep excavation through the hill. The condition favored the design of a line that would automatically create and maintain a vacuum to take the water over the hill without loss of head. Without any data to serve as a precedent, the diameters and grades at which the pipe was laid were designed to create this vacuum. The results show that a vacuum of about 10 ft. is automatically created and maintained whenever the pump is started.

A later and better opportunity for further application and use of these principles was presented in the design of three separate pipe lines that the

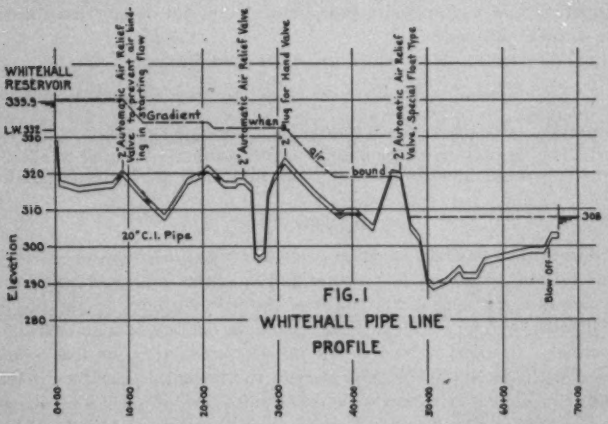

FIG. 1
WHITEHALL PIPE LINE
PROFILE

Boston Metropolitan District Water Supply Commission constructed in 1927 in the southern Sudbury drainage area. The principal facts in regard to these three pipe lines, referred to as the Whitehall, the Hopkinton and the Ashland lines, are as follows:

The Whitehall line is a gravity-flow 20-in. cast-iron pipe line about 6 740 ft. long from Whitehall Pond to a small diversion reservoir tributary to Hopkinton Reservoir, a drop of about 32 ft. (See Fig. 1.)

The Hopkinton line is normally a gravity-flow 30-in. cast-iron and reinforced-concrete pipe line, about 15 130 ft. long from Hopkinton Reservoir to Sudbury Reservoir, a drop of about 44 ft. For emergency use a branch line of reinforced-concrete pipe about 945 ft. long connects the Sudbury River at Cordaville with the Hopkinton line at a point about 8 130 ft. from its outlet at Sudbury Reservoir. Beyond this connection point, about 375 ft. nearer Sudbury Reservoir is located a pumping station containing a 250-h.p. motor-driven centrifugal pump which can either pump from the

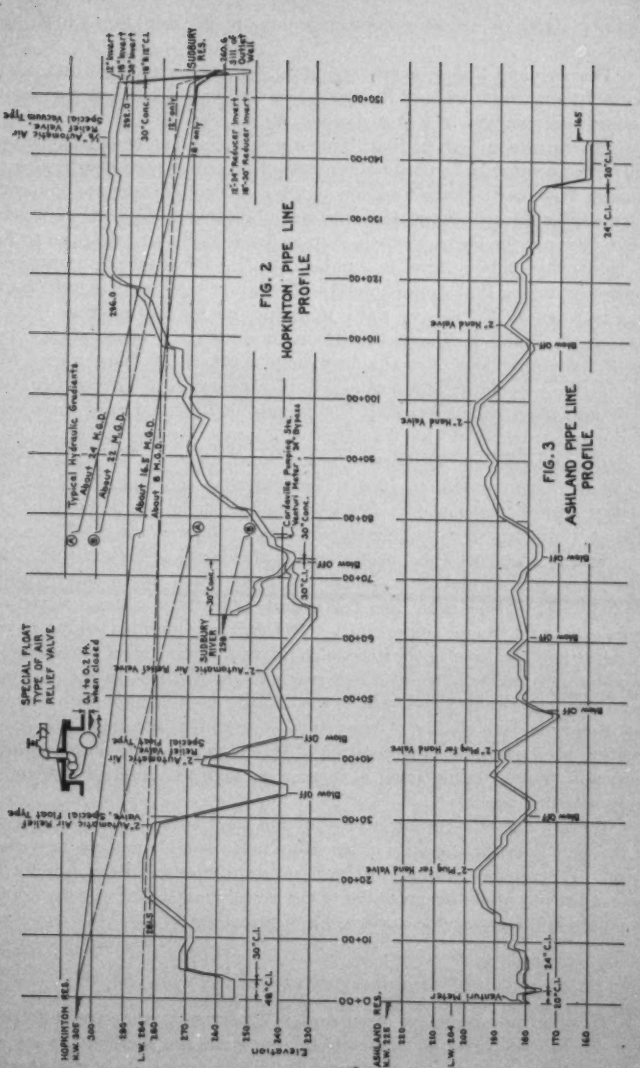
FIG. 2
HOPKINTON PIPE LINE
PROFILE
FIG. 3
ASHLAND PIPE LINE
PROFILE
SPECIAL FLOAT
TYPE OF AIR
RELIEF VALVE
0.1 to 0.2 Ft.
when closed
Typical Hydraulic Gradients
About 24 M.G.D.
About 22 M.G.D.
About 16.5 M.G.D.
About 8 M.G.D.
Blow Off
2" Hand Valve
2" Plug for Hand Valve
HOPKINTON RES.
ASHLAND RES.
SUDBURY RES.
Elevation
Natural Ground

Sudbury River or act as a booster pump on the Hopkinton line. (See Fig. 2.)

The Ashland line is a gravity-flow 24-in. cast-iron pipe line, about 14 400 ft. long from Ashland Reservoir to a well in the gatehouse at Framingham Reservoir No. 2, a drop of about 60 ft. (See Fig. 3.)

The entire southern Sudbury source is considered an emergency supply and would ordinarily be used only in case of an anticipated shortage in the existing Wachusett-North Sudbury supply. Such a shortage was imminent when construction of these pipe lines was undertaken, the level of Wachusett Reservoir, the District's main storage reservoir, having dropped to 32.3 ft. below the spillway level in January, 1927. The level of the reservoir continued during 1927 approximately 12 ft. lower than in 1926 until beginning in August, 1927, heavy rains, unusually heavy for that time of year, insured the filling of Wachusett Reservoir during the next spring. As a result these pipe lines from the emergency southern Sudbury source were not operated except for test purposes until January, 1930. Thereafter, they were used effectively throughout the drought of 1930 and until completion of the construction of the Wachusett-Coldbrook section of the Quabbin Aqueduct enabled the District to begin the diversion of flood flows from the Ware River in March, 1931. The continued shortage of rainfall has made it necessary to continue their use, pending the completion of the Commission's program of extending the aqueduct tunnel to the Swift River.

Similar problems were encountered in the design of each of these three pipe lines, and all had to be located at grades well up to the hydraulic gradient. The Hopkinton line, particularly, presented a great variety of problems and a unique opportunity to effect a considerable saving in cost by selecting a location for the line which involved counting on the automatic creation and maintenance of a high vacuum in the line. In fact it would have been practically impossible to accomplish the diversion from Hopkinton Reservoir into Sudbury Reservoir by anything like the method and location used except by the creation of this high vacuum. The line automatically creates within itself a vacuum as high as 23 in. of mercury, a truly startling result.

We will consider certain principles and their application in the design of the pipe lines, particularly of the Hopkinton line for reasons previously given. The profiles as illustrated are accurate in small detail but for the sake of brevity minor irregularities in the invert grade have been overlooked in places in discussing the application of principles.

The Determination of Invert Grades.

If, as in the case under discussion, it is necessary or desirable to lay a pipe line at an elevation higher than the hydraulic gradient under any condition of operation, it is obvious that, on account of the partial vacuum in the pipe, air relief valves cannot be counted on to eliminate air pockets.

Furthermore, even where the pipe is, as usual, below the hydraulic gradient, it is obviously desirable, from the standpoint of economy and reliability of operation, to require a minimum number of air relief valves. To this end one of the first considerations in determining the profile is to see that in as many cases as possible the slope of the invert grade downstream from all high points in the line is not materially greater than the slope of the hydraulic gradient, for a critical condition of low flow before the air pockets under consideration have had time to be eliminated.

In this particular consideration the only thing that matters is the slope of the pipe. Its elevation relative to the hydraulic gradient does not matter, since what we call the hydraulic gradient is the gradient at atmospheric pressure, and there is a similar and parallel gradient for every other pressure. The water in the closed pipe does not know, so to speak, what the pressure is outside.

In case the pipe is parallel to the hydraulic gradient, it is level as far as the travel of surface air bubbles is concerned (the bottom of every air bubble is sloped parallel to the hydraulic gradient); and the designer, knowing the hydraulic gradient and the velocity in the pipe available for carrying along air bubbles, can decide what maximum slope he will allow at any point to avoid the necessity of air relief valves. It should be noted that this statement refers only to the slopes falling in the direction of flow from the high points; the slopes approaching the high points from upstream, regardless of their length or relation to the hydraulic grade can be as steep as the topography demands, even vertical, without in any way contributing to the formation of air pockets.

After the preliminary layout, the selection of the proper grades downstream from the high points can be made by making adjustments in the depth of cuts and fills or even by altering the location of the line. This should always be a major consideration in first locating the line. The downstream end of the Hopkinton line was moved a considerable distance from the public highway onto a private right of way to make it possible to secure such a long flat slope downstream from the critical high point. As another example, the writer had occasion recently to locate a large sewer outfall beneath a ship channel. Here air valves were impracticable but a location was selected where the down grade approaching the channel location, was not steep, the steepness of the slope up on the far side of the channel being immaterial.

Referring to the profile of the Hopkinton 30-in. line, Figure 2, in which the normal velocity is a little over 4 ft. per sec. and the slope of the hydraulic gradient about 0.2 per cent., the down grades of the invert in such critical sections as previously referred to were kept as flat as 0.35 per cent. in the long stretches particularly those under a high vacuum. In the shorter stretches 1.0 per cent. was allowed, there being one length of 450 ft. at this slope. The shorter the stretch involved, the greater the slope which may safely be used since the worst result from a failure of the water to

carry along the air is a steepening of the hydraulic gradient for a short distance only.

In the Whitehall 20-in. line, Figure 1, in which the normal velocity is a little over 4.5 ft. per sec. and the slope of the hydraulic gradient about 0.45 per cent., the down grades of the invert in such critical sections as previously mentioned could not be kept flatter than 2.0 to 2.25 per cent. in fairly long stretches.

In the Ashland 24-in. line, Figure 3, in which the normal velocity is a little over 4.5 ft. per sec., and the slope of the hydraulic gradient about 0.35 per cent., the down grades of the invert in such critical sections as previously referred to could not be kept flatter than 2.0 to 3.0 per cent. in fairly long stretches.

In the case of these steeper slopes, materially steeper than the hydraulic gradient, provision was made by drilling and tapping a 2-in. hole in the top of the pipe at the high point or upstream end of each of these long down grades for inserting small hand valves for use in investigating the accumulation of air during tests of the operation of the line. These tests showed that it was unnecessary to install air relief valves at these points, and the holes were plugged in such manner that they could be used for air vents at any future time, if necessary. In the longer stretches where the pipe would be somewhat above the hydraulic gradient under some conditions of operation, on both the Whitehall and the Ashland lines, hand valves were left in place in order to permit continuing experimental work or venting accumulations of air that might interfere with starting the flow in the line.

In stretches like those previously mentioned in which it is difficult to obtain a sufficiently flat down grade, it is better to have the steepest part of this slope near the upstream end of the reach and the flattest part near the downstream end. Then in case the velocity in the pipe is slowed down for any reason to such an extent that at these steepest slopes the water is barely capable of carrying along the air, the loss of head due to a possible air pocket, reduced water area and steep gradient, is confined to a relatively short length of pipe near the upstream end of this down grade. If under similar circumstances the steepest invert grade were located near the downstream end of the slope the air bubble would ultimately extend back to the top of the hill, and a much greater length of pipe would be affected with consequent greater loss of head.

Having revised the location of the line, modified the depth of cuts and made such other changes as necessary to obtain the flattest possible down grades from high points in the line, there will undoubtedly still remain high points from which the down grade is too steep to count on the prevention of air pockets in the manner previously mentioned. In the case of such of these high points as are located at all times below the hydraulic gradient, good practice dictates the installation of automatic air relief valves of standard type designed to remain open so long as there is any air in the top of the pipe and otherwise to close so as to prevent the discharge of water

from the pipe. In the case of any of these high points that lie somewhat above the hydraulic gradient under certain conditions of operation, it is necessary to install an air relief valve of special design which will not only perform the functions of the standard valve as previously stated, but which will also remain closed to prevent air being sucked into the pipe under a partial vacuum.

One such valve of special design was installed on the Whitehall line at a point where the top of the pipe might be at times about 4 ft. above the hydraulic gradient and two such valves were placed in the Hopkinton line where the top of the pipe might be at times from 2 to 3 ft. above the hydraulic gradient. The design used for this purpose is shown in Figure 2, and consists of a 2-in. float cock just beneath the manhole cover set to close with the rise of water to near the top of the pipe and to remain closed until the water level drops from 0.1 to 0.2 ft. below the top of the pipe, and above the manhole cover one 2-in. globe valve and one 2-in. check valve, all three valves being in series. Although the float shuts the valve before the air is entirely expelled, the high velocity in the practically full pipe is counted on to slowly but nevertheless effectively remove the remaining small quantity of air in the top of the circular cross section and hence positively close the valve. The float maintains a positive closing pressure, and even though air may later leak back slowly through the valve into the chamber under the manhole cover, this air also is effectively removed by the water in the full pipe. This design was used to secure greater capacity in a valve of this type than could be obtained commercially in a single compact design.

Later a small commercial ¾-in. valve of this same type, that is one to eject the air under pressure but prevent its entrance under vacuum, was installed near the downstream end of the Hopkinton line at the point of normal high vacuum, not for the purpose of improving the operation but to allow the air to escape gradually when the pipe was out of operation for long periods with the main shut-off valves closed at the Sudbury end so that the entire line would be full of water upon starting again and so that it would be unnecessary for the automatic vacuum-creating device described later to remove such a large quantity of air — from about 1½ miles of the pipe.

Prevention of Air Binding.

Having first determined the minimum number of air valves required for proper operation after the flow has been started, it is next necessary to determine whether air relief valves must be added at other points in order to prevent air binding when starting flow. If a pipe line has enough high points of marked extent, that is, enough ups and downs, it can become air bound to such an extent that continuous flow cannot be started under the maximum head available. To determine just how many air valves are necessary to prevent this condition an analysis should be made of the profile of the pipe line to show positively the worst condition of air binding.

For example, on the Whitehall pipe line, Figure 1, two air relief valves were found to be necessary at the upstream ends of steep slopes, at Stations 25 + 50 and 46 + 64, but with only these two air relief valves and with the line empty, or as nearly empty as it can ever be, water turned into the pipe at the upstream end would build up air pressures successively higher and higher at the various high points from atmospheric pressure at Station 47 + 50 to the maximum available pressure at the upstream end. There would, therefore, be no head available for starting continuous flow throughout the line. The water levels in the pipe and the corresponding hydraulic gradient for this air bound condition are shown on the profile. The computations consist merely of a static balancing of the various air and water columns, starting with whatever volume of free air is trapped upstream from each low point.

This study pointed to the necessity of an additional air relief valve at Station 9 + 20. It was also considered desirable to add a small hand valve at Station 31 + 00 to permit investigating the accumulation of air at this point if desired.

In a similar way, every pipe line should be analyzed to determine the possible necessity of additional air relief valves solely for the prevention of air binding, after the minimum number of such valves has first been determined to fit the requirements of the invert grade.

The Automatic Creation and Maintenance of a Vacuum.

In the case of the relief valves of special design shown on Figure 2 and installed at the various points previously referred to, these special features have no part in filling the pipe with water, but are designed only to assist in holding it full. For example, these features are not required when flow is first started in the pipe, but as the reservoir is gradually drawn down and the hydraulic gradient lowered they help maintain a slight vacuum at the high point. Excessive leakage of air, accidental or otherwise, past such a valve might result in failure to hold the vacuum. Hence this type of valve would not accomplish its purpose satisfactorily unless installed at an elevation always lower than the expected minimum level of the reservoir or source of supply, so that in case of such failure of the vacuum, flow would still be maintained, although somewhat reduced in volume, and the valve would remain in position again to eject the air under sufficient reservoir fluctuation, and help to hold the vacuum anew.

The creation and maintenance of a vacuum can, however, be counted on regardless of air leakage into the pipe. This was done at the Sudbury Reservoir end of the Hopkinton line. The design there employed results in the ejection of the air from the pipe and the creation of a high vacuum, as high as 23 inches of mercury, and holds it even after the level of the Hopkinton Reservoir source has dropped considerably below the high point in question and in spite of leakage of air into the pipe and extraction of air

from the water itself under the high vacuum. About 4 600 ft. of the pipe are normally under vacuum at and near this point. The pipe at this point is of pre-cast concrete containing a thin welded steel cylinder embedded in the concrete. The joints are packed with fibre-filled lead gaskets. A continual stream of air bubbles is required to be ejected. Undoubtedly this air comes largely from the water itself.

The crown of the 30-in. line at the high point near the discharge end is only 6.5 ft. below the level of high water in Hopkinton Reservoir.

The booster pump at Cordaville, previously mentioned, can be used in case it should ever be required to start the flow in the line at a time when the Hopkinton Reservoir source is at a lower elevation than this high point near the Sudbury Reservoir or discharge end.

The pipe downstream from the high point is laid on a slope not exceeding 0.35 per cent. as previously described. This slope is continued to the brow of a hill overlooking the reservoir channel 35 ft. below and only 230 ft. distant. Here the 30-in. pipe divides into two smaller pipes, one 18-in. and one 12-in., which run on a steep grade down the hill 230 ft. to a well 6 ft. in diameter, where they are controllable with valves for regulating the flow in them. Velocity head in the small pipes is recovered by enlarging the 12-in. diameter to 24 in. and the 18-in. diameter to 30 in., respectively. The enlargements and the entry into the well are submerged below the level of the water as it flows out of the well into the channel leading to the Sudbury Reservoir.

The 30-18-12-in. branch at the brow of the hill is designed with the smaller outlets sufficiently elevated so that much of the air in the 30-in. portion is expelled by the simple process of filling the pipe with water before automatic ejection of air starts, and it is so shaped that the jets of water tend to mix with the air that is left.

The final ejection of air is accomplished by a combination of thorough mixing of air and water and by high velocity in the small pipes. The economy lies in the fact that these small pipes are very short and that their hydraulic gradient though steep is not nearly as steep as the slope of their invert grade. While the vacuum is being built up and before the pipe is filled, the mixing of air and water is accomplished by a hydraulic jump, which occurs inside the pipe at the end of the air pocket; after the pipe is filled, the mixing is assisted by sharp bends at the 30-18-12-in. branch, as previously described.

Valves on both the 18-in. and 12-in. pipes near the well at the bottom make it possible to use the 12-in. pipe only, the 18-in. pipe only, or both together. The 18-in. pipe alone was designed for normal operation with gravity flow from Hopkinton Reservoir. As a matter of fact, the vacuum is maintained so easily that the 18-in. and 12-in. pipes when open together operate satisfactorily under normal gravity flow, although designed to be used in parallel only at the higher rate of flow resulting from the use of the booster pump. The possibility of using the 12-in. pipe alone was provided

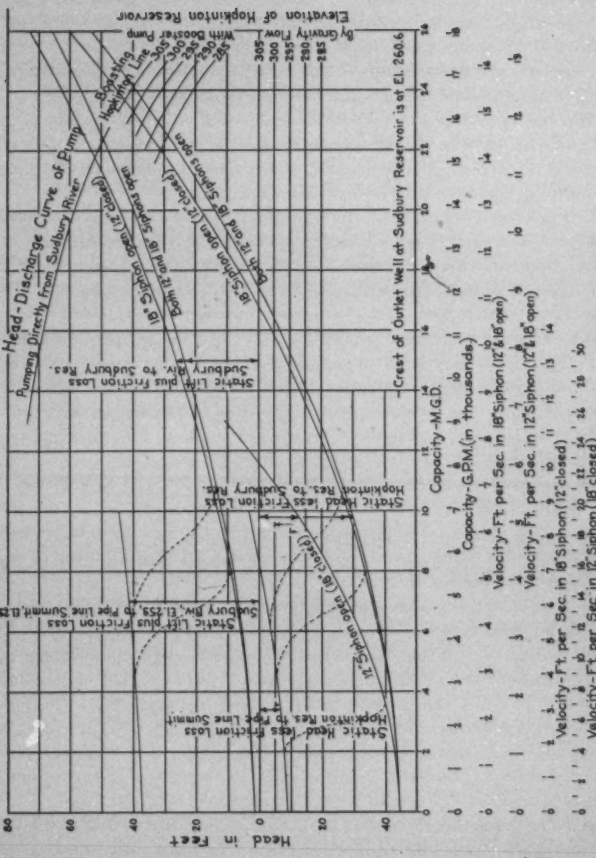
Elevation of Hopkinton Reservoir
By Gravity Flow
With Booster Pump
Head in Feet
Head-Discharge Curve of Pump
Pumping Directly from Sudbury River
Boosting Hopkinton Line
Crest of Outlet Weir at Sudbury Reservoir is at El. 260.6
Capacity-M.G.D.
Capacity-G.P.M.(in thousands)
Velocity-Ft. per Sec. in 18"Siphon(12"&18" open)
Velocity-Ft. per Sec. in 12"Siphon(12"&18" open)
Velocity-Ft. per Sec. in 18"Siphon(12"closed)
Velocity-Ft. per Sec. in 12"Siphon(18"closed)
Static Lift plus Friction Loss Sudbury Rv. to Sudbury Res.
18"Siphon open (12"closed)
Both 12" and 18" Siphons open
Both 12" and 18" Siphons open
18"Siphon open
Static Head less Friction Loss Hopkinton Res. to Sudbury Res.
12"Siphon open (18"closed)
Static Lift plus Friction Loss Sudbury Rv. El.253. to Pipe Line Summit El.336
Static Head less Friction Loss Hopkinton Res. to Pipe Line Summit

FIG. 4.—PERFORMANCE OF HOPKINTON PIPE LINE SIPHON.

in case its higher velocity should at any time be found necessary in starting the flow. Minor details of the design include a small manually operated valve on a cross connection between the high points of the 18-in. and 12-in. pipes to equalize the two vacuums, and to prevent the possibility of one pipe doing all the work when two full pipes are wanted.

The critical requirements of velocity sufficient to carry along the mixture of air and water are those in the case of starting the flow when the reservoir at the source has been pulled down to nearly the elevation of the high point of the siphon. The hydraulics of this situation are illustrated by the performance curves of Figure 4, which show how the flow in this Hopkinton line is governed. On this chart there are two groups of head-capacity curves running from the lower left to the upper right corner of the diagram. The curves in the upper group show the loss of head in the line from the Sudbury River to the Sudbury Reservoir which includes the branch connection at Cordaville and approximately half of the Hopkinton line, and are for use only in the case of pumping directly from the Sudbury River when the flow from Hopkinton Reservoir is shut off by closing a valve in the main line. The curves in the lower group show the loss of head in the entire main line from Hopkinton Reservoir. The short nearly horizontal lines at the left are for the length of pipe measured only to the high point of the siphon where the vacuum has to be maintained and the longer lines include the friction in the 18-in. and 12-in. automatic siphon pipes connecting the pipe line summit with the outlet well at Sudbury Reservoir.

The dotted curves that connect the shorter lines to the longer ones near the left of the chart are intended to illustrate approximately what takes place when the siphons are automatically primed. For example, when the flow is first started in the line, the governing head is that measured to the pipe line summit, but when the velocity reaches about 4 ft. per sec. in the priming pipes, the automatic ejection of air commences and the resulting vacuum increases the available head. The performance indicated by the complete drop to the lower set of curves corresponds to practically complete ejection of the air which is the case, for example, whenever the pump is in use.

Let us consider the case of gravity flow from Hopkinton Reservoir assuming that it is desired to start such flow when the reservoir is 5 ft. below the spillway or at elevation 300. This reservoir elevation is found in a scale at the right of the chart, and the proper elevation, 300, may be followed across the chart to show that under this condition there would be only about 4 ft. head available to start the flow over the summit. The curves whose drop represents the priming of the siphons have been placed upon the chart as a result of various tests in priming under low head. They indicate that with the reservoir as low as elevation 300 the opening of both 12-in. and 18-in. siphons would result in too low a velocity to insure their priming and the discharge could not be counted on to exceed 5 m.g.d. unless the booster pump was started. It would then, if the pump was first

run long enough to get rid of all air in the line, be maintained by gravity flow at about 16½ m.g.d. Similarly, the curves show that with the 12-in. gate closed and the larger 18-in. one opened, and with Hopkinton Reservoir at elevation 300, automatic priming could just be counted on to gradually increase the flow to a maximum of a little less than 16 m.g.d. with all the air ejected. Similarly, with the 18-in. gate closed and the smaller 12-in. gate open, the line would be safely primed and the flow would gradually increase to a maximum of a little less than 11 m.g.d. with all the air ejected.

The condition of maximum freedom from air throughout the entire length of the line is, of course, more nearly attained in the case of the higher rates of flow. For example, experience in operating the line indicates that

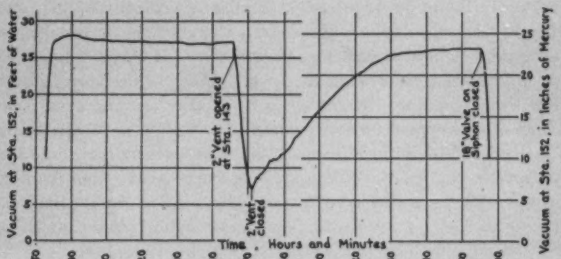

FIG. 5
HOPKINTON PIPE LINE SIPHON
TEST OF VACUUM RECOVERY, Oct. 10 '27

under very unfavorable conditions for gravity flow, namely Hopkinton Reservoir drawn down 10 to 15 ft., the booster pump not in operation, and incidentally with the pipes 6 years old as they now are, the continuous rate of flow that can be counted on throughout the season is not less than 80 per cent. of that shown on the chart by these curves for new pipe and complete freedom from air. As the rate of flow increases, it approaches that indicated by the curves, and in the case of maximum flows with the pump in operation it is practically the same as shown on the chart, except for any allowance that must be made for the aging pipe. As a result of last season's experience, rates obtainable in practical operation may be taken at somewhat over 21 m.g.d. pumping from Sudbury River, 14 m.g.d. gravity flow from Hopkinton Reservoir using the 18-in. siphon with the reservoir well up and 8 m.g.d. using the 12-in. siphon with the reservoir considerably depleted.

The ability of the siphons to prime themselves automatically is well illustrated in Figure 5, which shows the results of a test made by opening wide a 2-in. valve at the summit of the line to break the vacuum of almost

24 in. of mercury. The valve was left open five minutes during which time the vacuum dropped to about 6 in. The valve was then shut and the air which had been allowed to enter the line was automatically ejected, the vacuum being built up again to 22 in. in 37 minutes, then continuing to build up at a slower rate.

Figure 6 is a photograph of the outlet of the Hopkinton Line in full

FIG. 6.— DISCHARGE FROM HOPKINTON PIPE LINE SIPHON.

operation, showing the water with its entrained air discharging over a weir in the side of the 6-ft. outlet well.

In the preparation of this paper the writer has received considerable assistance and many valuable suggestions from Lawrence M. Gentleman, Assistant Engineer in the Designing Division of the Metropolitan District Water Supply Commission, who has spent considerable time on the details of the design of the Whitehall, Hopkinton and Ashland pipe lines.

All three pipe lines in the southern Sudbury area were built as a part of the program of the Metropolitan District Water Supply Commission, of which Frank E. Winsor, Past President of the New England Water Works Association, is Chief Engineer.

MANGANESE IN IMPOUNDED WATER SUPPLIES.

BY ROBERT SPURR WESTON* AND ATTMORE E. GRIFFIN.†

[*Read September 30, 1931.*]

Manganese, which is grouped with iron in chemical classifications, rarely occurs in impounded waters in appreciable quantities. Still more rarely is its determination made. In the stagnant bottoms of certain deep reservoirs, a condition which favors the bacteriological decomposition of sulphates with the production of hydrogen sulphide, manganese is often present but usually in less amount than iron. In a few reservoirs, however, the reverse is true.

Manganese is more frequently found in ground than in surface waters, and it is undoubtedly true that in many waters its presence would have attracted little attention had it not been for its effect on the ortho-tolidine test, used so frequently to control treatment of water with chlorine. Forsberg[1] showed that in connection with this test manganese in surface waters reacted like chlorine. Unfiltered waters gave a positive ortho-tolidine reaction, but these same waters filtered through paper to remove the manganese did not so react. In New Jersey, Forman[2] experienced considerable difficulty because of this interference of manganese with the ortho-tolidine test. He found that, when varying amounts of manganese were present, the determination of the "chlorine blank"‡ was interfered with and consequently that the proper dosing of the water with chlorine was made more difficult.

Excessive occurrence of manganese in ground water has been observed, notably in Breslau, Germany[3] in the spring of 1906, but such occurrence in waters of this class is a subject apart from this paper.

Table 1 shows the amounts of manganese that have been determined in surface waters.

Since March 8, 1930, when the Wanaque Reservoir of the North Jersey District Water Supply Commission was placed in service, the authors of this paper have studied the effect of the soil at the bottom of this reservoir upon the character of the water stored in it.

The Wanaque Reservoir. The Wanaque Reservoir has been ably described in a paper presented to this Association by Arthur H. Pratt[4]. This reservoir is located in the New Jersey highlands near the New York line; it is designed to supply 100 m.g.d. to Newark, Paterson and other municipalities in populous northeastern New Jersey. The consumption at present averages about 75 m.g.d.

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*Of Weston & Sampson, Consulting Engineers.

†With Wallace & Tiernan. Formerly with Weston & Sampson in charge of the Wanaque laboratory of the North Jersey District Water Supply Commission.

‡The "chlorine blank," or chlorine demand, is the amount of chlorine required to saturate the dead organic matter and other constituents in water. The excess above this is the residual chlorine, available as a bactericide.

TABLE 1.—MANGANESE CONTENT OF SURFACE WATERS.

Locality	Manganese p.p.m.
Amherst, Mass.*	0.01
Amsterdam, Holland	0.30
Baltimore, Md.	0.20 or less
Barberton, O.	0.90
Boonton, N. J.	0.04
Boston, Mass. (Metropolitan)*	0.01
Branchville, N. J.	0.08
Brockton, Mass.*	0.01
Burlington, N. J.	0.10
Butler, N. J.	0.04
Columbus, Ohio (Scioto River)	0.22
East Liverpool, O. (Ohio River)	0.88
Jersey City, N. J.	0.04
Lake Enka, N. C.	0.70
Marlboro, Mass.*	0.04
New Bedford, Mass.*	0.03
New York City (Croton System)	0.03 to 0.10
Ogdensburg, N. J.	0.06
Oakland and Berkeley, Cal.	less than 0.10
Phillipsburg, N. J.	0.04
Providence, R. I.†	0.15 to 0.75
Rahway, N. J.	0.03
Springfield, Mass.*	0.14
Sussex, N. J.	0.08
Wanaque, N. J. (Wanaque Reservoir)	0.00 to 2.5
Youngstown, O. (Meander Creek)	0.22

The reservoir is in a favored location, and is an excellent engineering work. It has a catchment area of 94.4 sq. miles. Its area is 2 300 acres at elevation 300.3, and its capacity is 28 010 m.g. In the catchment area contributing to the reservoir are Greenwood and Sterling Lakes, the former receiving the run-off from 27.1 sq. miles. The yield of the area is high, over 1.1 m.g.d. per sq. mile; and the storage ratio in the reservoir itself is 0.70, equivalent to over eight months, or if storage in Greenwood Lake on its principal tributary is included, the storage ratio is 1.0.

The valley of the Wanaque River is one which has been scoured by glacial action, and then re-filled with deposits. At the dam site, these deposits reach a depth of 110 ft. The terrain is hilly, and the several feeders drain areas sparsely covered with soil. Only 1.5 per cent. of the area flooded consists of swamp. From the view-point of this paper the important characteristic of the area within the watershed is the presence of several mines, one of them Peter's Mine, which has been worked intermittently for many years.

The reservoir began to fill in March, 1928; filling was complete on March 4, 1929; water was first delivered on March 8, 1930 to two, and in

*Analyses by courtesy of Mr. H. W. Clark, Chief Chemist, Massachusetts Department of Public Health.

†Analyses by courtesy of Mr. E. L. Bean, Chemist, Providence Water Works.

July of this year to all of the municipalities participating in the project. The average composition of the reservoir water is given in Table 2.

TABLE 2.—AVERAGE COMPOSITION OF WANAQUE WATER.*

Constituent.	Parts per Million. 1930	As discharged.	At surface of reservoir.
Turbidity	5	2	3
Color (true)	13	11	9
Color (apparent)	21	15	14
Oxygen consumed	3.6	3.3	....
Ammonia Nitrogen	0.015	0.029	....
Albuminoid Nitrogen	0.152	0.143	....
Nitrite Nitrogen	0.001	0.003	....
Nitrate Nitrogen	0.05	0.04	....
Chlorides	2.0	1.8	....
Hardness	20	23	....
Alkalinity	14	15	....
Total Residue on Evaporation	40	45	....
Carbon dioxide	1.5	1.1	....
pH value	7.0	6.8	6.9
Iron	0.16	0.36	.10
Manganese	.....	0.23	.17

In 1930 the manganese in the delivered water ranged from 0.0 to 2.4 p.p.m.†; in 1931 it ranged from 0.09 to 1.56 p.p.m. and averaged 0.23 p.p.m.

These results show the water to be a surface water of exceedingly high grade,— low in color, soft, and with promise of being excellent in appearance after the reservoir has aged. It was predicted by one of us that the color of this water would average 12 p.p.m. This prediction has been bettered already,— the true color (the color after filtration through paper) for 1931 averaging 11 p.p.m. So far, the results for 1932 indicate a still lower average.

Owing to its low mineral and organic content, the water exerted a solvent action on the distribution systems of the towns supplied with it, and also upon the old deposits therein. Consequently, it was found necessary, early in September, 1930, to add hydrated lime to the water to raise the pH value from an average of 6.8 to one of about 8.8, to prevent corrosion and "red water" troubles.

The delivered water has always been chlorinated, and near the end of the first summer it was found that manganese interfered with the control of this process. This led to the determination of manganese, the variations in the amounts of which found in the water are strikingly shown in Figures 1, 3 and 4 and in Table 3.

In 1930, from May until late December, daily ortho-tolidine tests were made on the raw water, and "blanks" or values of "chlorine demand"

*Supplied by courtesy of J. William Griffin, Superintendent of Operations.
†At bottom of reservoir July 24, 1930.

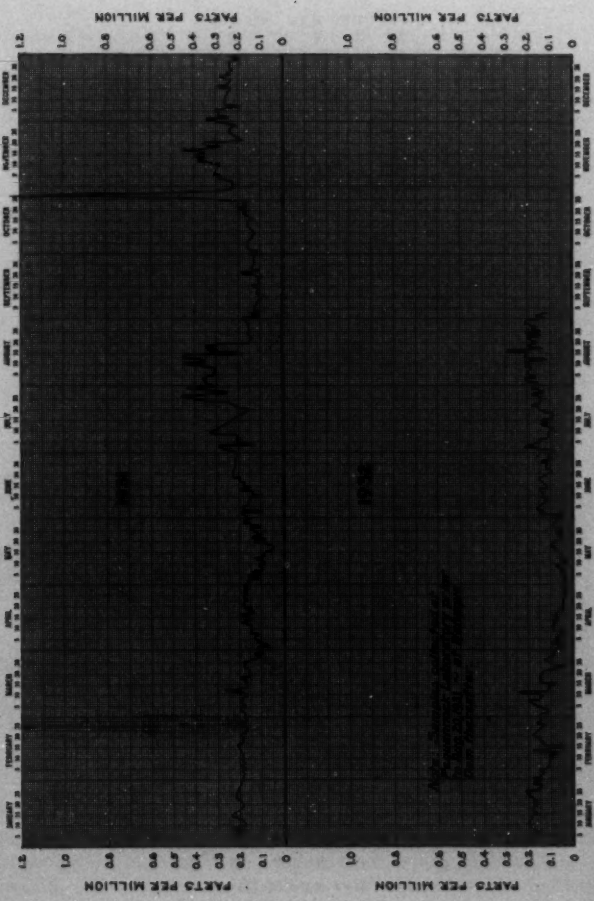


FIG. 1. — WANAQUE RESERVOIR. MANGANESE IN DELIVERED WATER, 1931 AND 1932.

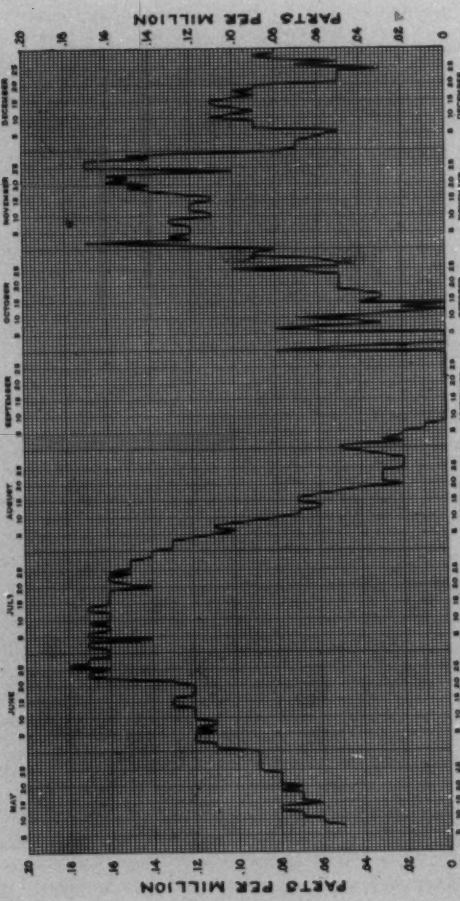


FIG. 2.—WANAQUE RESERVOIR, 1930. ORTHO-TOLIDINE TEST BLANK READING, MAY TO DECEMBER, 1930.

were obtained. These ranged from 0.0 to 0.18 p.p.m. (See Fig. 2.) They were obtained by the method commonly advised by the New Jersey Department of Health and other authorities, and were not questioned. However, they gave a false idea of security because it was believed until then by us that this method would include manganese in the "blank" just as it

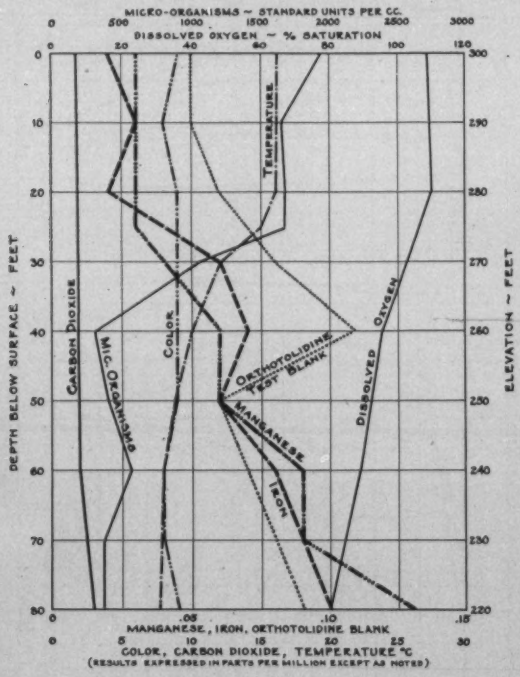


FIG. 3. — WANAQUE RESERVOIR, ANALYSES OF DEEP SAMPLES, MAY 23, 1932.

did organic and other matters consuming chlorine. Really, however, the water contained considerable manganese in combinations which did not react with the ortho-tolidine. Consequently, when chlorine was added, some of it combined with the manganese and was lost, with the result that the figures for residual chlorine in the delivered water included manganese as well as chlorine and led to the belief that sufficient chlorine had been added, when the contrary was true. Fortunately, the raw water, even then, was of high quality and one not ordinarily thought to require treatment with chlorine.

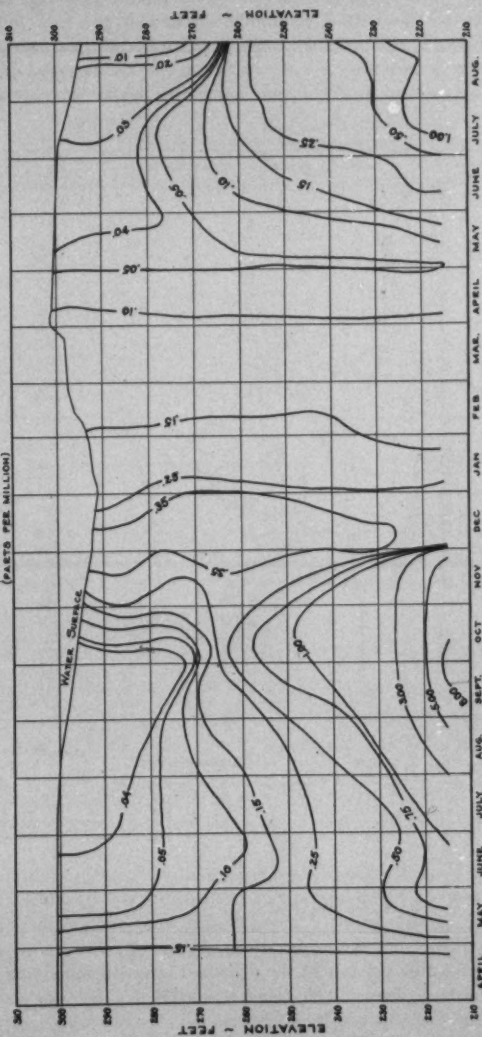

ELEVATION ~ FEET
(PARTS PER MILLION)
Water Surface
1931
1932
APRIL MAY JUNE JULY AUG. SEPT. OCT NOV DEC JAN FEB MAR. APRIL MAY JUNE JULY AUG.

FIG. 4. — WANAQUE RESERVOIR. MANGANESE CONTENT AT VARIOUS DEPTHS, 1931 AND 1932.

TABLE 3. — WANAQUE RESERVOIR. MANGANESE DETERMINATIONS AT VARIOUS DEPTHS.

~ PARTS PER MILLION ~

Depth	1930		1931																						Depth
	7/17	7/24	4/30	5/27	7/3	7/21	8/11	8/24	9/3	9/14	9/23	9/28	10/8	10/13	10/19	10/26	11/9	11/16	11/18	11/23	11/30	12/7	12/12	12/21	
Surface	.20	.00	–	.05	.04	.03	.04	–	.03	–	.03	–	.03	–	–	–	–	–	.28	–	.41	–	.36	.25	Surface
10 ft.	–	–	–	.06	.03	.03	.04	–	.04	–	.03	–	.03	–	–	.15	.33	.36	.25	.40	.34	.38	.35	.33	10 ft.
15 "	–	–	.15	–	–	.04	–	.04	–	.00	–	.00	–	.06	.00	–	–	–	–	–	–	–	–	–	15 "
20 "	–	.05	–	.06	.04	.06	.04	–	.04	–	.03	–	.03	–	–	–	–	–	.26	–	–	–	.36	–	20 "
25 "	–	.05	–	–	–	–	.08	–	.11	–	.03	–	.04	–	–	–	–	–	.25	–	–	–	–	–	25 "
30 "	.05	.06	–	.08	.05	.08	.11	–	.15	–	.23	–	.15	–	–	.14	.29	.38	.27	.35	.43	.50	.35	.38	30 "
35 "	–	–	.16	–	–	–	–	.31	–	.13	–	.27	–	.48	.15	–	–	–	–	–	–	–	–	–	35 "
40 "	.15	.14	–	.19	.11	.13	.14	–	.24	–	.82	–	.75	–	–	–	–	–	.45	–	–	–	.35	–	40 "
50 "	.53	.36	–	.20	.24	.29	.25	–	.45	–	.88	–	1.13	–	–	–	–	–	.50	–	–	–	.36	–	50 "
60 "	–	1.90	–	.29	.25	.31	.58	–	.55	–	1.38	–	1.40	–	–	–	–	–	1.00	–	.36	.35	.35	.24	60 "
70 "	1.69	–	.16	.48	.31	.53	1.06	1.31	.58	1.44	2.25	1.60	3.00	3.44	.63	2.38	2.13	.88	5.63	.80	–	–	.31	–	70 "
75 "	1.88	2.40	–	–	–	–	–	–	–	–	–	–	5.63	–	–	–	–	–	5.00	–	4.5	–	–	–	75 "
80 "	–	–	–	.90	1.31	2.00	4.25	–	4.75	–	8.75	–	–	–	–	–	–	–	–	–	–	–	–	–	80 "
Average	.72	.62	.16	.26	.27	.35	.66	.55	.69	.52	1.22	.62	1.22	1.33	.26	.89	.92	.54	1.39	.52	.40	.41	.35	.30	Average

Depth	1932																								Depth
	1/4	1/12	2/1	2/8	2/15	2/27	4/12	5/4	5/16	5/23	6/1	6/7	6/15	6/20	6/27	7/5	7/11	7/18	7/26	8/1	8/8	8/15	8/22	8/29	9/1	
Surface	–	.21	.19	–	.06	.16	.08	.05	.05	.03	.01	.04	.04	.04	.04	.04	.03	.03	.01	.04	.03	.03	.01	.01	.01	Surface
10 ft	.23	.23	.20	.11	.08	.15	.08	.04	.05	.03	.01	.03	.04	.03	.04	.04	.03	.04	.01	.04	.03	.03	.01	.01	.01	10 ft.
15 "	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	.01	15 "
20 "	–	.23	.15	–	.09	.16	.09	.05	.05	.03	.01	.04	.06	.04	.06	.05	.04	.04	.04	.04	.03	.04	.01	.01	.01	20 "
25 "	–	–	–	–	–	–	–	–	–	.03	.03	.05	.05	.04	.05	.08	.08	.05	.06	.06	.04	.04	.01	.01	.03	25 "
30 "	.35	.21	.18	.18	.08	.15	.09	.04	.04	.04	.04	.09	.05	.04	.06	.09	.05	.10	.13	.08	.04	.04	.03	.03	.03	30 "
35 "	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	.28	.21	.40	.09	35 "
40 "	–	.20	.18	–	.09	.16	.09	.04	.06	.06	.05	.06	.14	.14	.09	.14	.15	.16	.18	.14	.30	.30	.31	.48	.36	40 "
60 "	–	.21	.20	–	.11	.15	.11	.05	.06	.06	.08	.13	.13	.13	.18	.25	.27	.28	.31	.19	.20	.34	.30	.48	–	50 "
60 "	.24	.23	.15	.10	.10	.16	.11	.06	.09	.09	.10	.18	.15	.20	.23	.21	.33	.35	.29	.43	.43	.48	.33	.53	–	60 "
70 "	–	.24	–	–	–	–	.09	.04	.09	.09	.10	.18	.18	.20	.21	.35	.35	.50	.55	.45	–	.88	–	1.13	–	70 "
75 "	–	–	.09	–	–	–	–	–	–	–	.21	–	–	–	–	–	.40	1.00	–	–	1.13	–	.88	–	–	75 "
80 "	–	–	–	–	.08	.18	.11	.05	.10	.13	.08	–	.40	.21	.44	.69	–	–	–	–	–	–	–	–	–	80 "
Average	.27	.22	.17	.13	.12	.16	.09	.05	.07	.06	.05	.10	.12	.11	.14	.19	.17	.26	.18	.16	.25	.22	.21	.31	.07	Average

The relation between the iron, manganese, ortho-tolidine test and the temperature in samples collected on May 23, 1932 from various depths in the Wanaque Reservoir is shown in Figure 3.

Manganese in Feeders. The amounts of manganese found in the main feeders of the reservoir are given in Table 4.

TABLE 4.—MANGANESE IN WANAQUE RESERVOIR FEEDERS, 1931.

Feeder.	Parts per Million. Maximum	Minimum	Average
Blue Mine Brook	0.00	0.00	0.00
West Brook	0.04	0.00	0.01
Cupsaw Brook	5.63	0.06	1.98
Erskine Brook	....	....	0.16
Ringwood Creek	0.05	0.03	0.04
Wanaque River	0.01	0.01	0.01
Reservoir	1.56	0.09	0.23

While much manganese came from Cupsaw Brook at times, its catchment area is relatively small, and the increase in manganese in the reservoir cannot be accounted for thereby. This feeder has been dammed to form an artificial lake, around which high-class cottages are being built, and it is quite probable that the manganese in the brook water is higher now than it will be after the artificial lake has aged.

Distribution of Manganese. By referring to Figure 1, it can be seen that the manganese content of the reservoir water is highest when the water overturns, *i. e.* when the bottom water is brought to the surface. Depth samples collected when the reservoir was stagnant and stratified show high amounts of manganese, carbon dioxide, and some hydrogen sulphide in the samples from the lower or stagnant zone, as shown in Table 5.

TABLE 5.—COMPOSITION OF WANAQUE WATER AT VARIOUS DEPTHS, SEPTEMBER 3, 1931.

		(Parts per Million.)					
Depth below surface (ft.)	Temp. Deg. C.	Color Apparent.	Carbon Dioxide.	Hydrogen Sulphide.	Dissolved Oxygen (% Sat.)	Iron.	Manganese.
0	23.0	11	1.8	0.00	95	0.02	0.03
10	22.7	11	2.1	0.00	97	0.03	0.04
20	22.0	12	3.7	0.00	69	0.03	0.04
25	20.4	15	6.9	0.00	10.0	0.05	0.11
30	15.7	16	8.3	0.00	7.6	0.08	0.15
40	11.8	16	7.9	0.00	33	0.07	0.24
50	10.8	16	9.8	0.00	13	0.24	0.45
60	10.5	18	10.1	0.00	16	0.18	0.55
70	10.0	18	8.1	0.05	Trace	0.28	0.58
81	9.1	45	12.0	0.25	0.00	1.28	4.75

Origin of Manganese. The presence of manganese in water is due to the same causes as the presence of iron, *i. e.* it is found as a component of the iron ore which lies beneath reservoirs like the Wanaque Reservoir, and is dissolved therefrom by the action of organic and mineral acids, notably carbonic acid. It is not unlikely that compounds of manganese with organic matter exist in swampy areas covered by the reservoirs. The feeders receive these just as they do organic compounds of iron.

In all probability the solution of manganese from the soil does not occur in the presence of oxygen, and Soehngen has clearly shown how it is due to anaërobic processes, especially those connected with bacterial decomposition and the production of hydrogen sulphide.

The solution of manganese generally occurs at the bottom of the reservoir during the stagnant period, as shown in Table 5, and is distributed throughout the reservoir during the overturn as shown in Figures 1 to 3. A solution of manganous bi-carbonate is difficult to oxidize; consequently, when the manganese is brought up from the bottom of the reservoir, it oxidizes but slowly. On the other hand, the iron oxidizes rather quickly and settles out of the water, playing an important part in its decolorization. This slow oxidation of manganese may explain why so much of it reaches the aqueduct, and why its ratio to the iron in the delivered water is so high.

When the manganese is in the form of an organic compound or in the form of sulphate, its oxidation by the oxygen dissolved in the water is still more difficult and according to von Wolzogen Kühr[5] may depend upon the growth of manganese bacteria and fungi. D. D. Jackson has described a species of *Crenothrix* (*C. manganifera*),[6] which plays a part in oxidizing manganese. Others have written about the action of these organisms, but chiefly with relation to iron or to manganese in ground water.

The experiences with the Wanaque water are in line with those of Dr. Frank E. Hale with the New York Croton water.[7] Normally the effluent of Croton Lake contains but 0.08 p.p.m. of manganese, but between September 1 and 8, 1925, an unexpected drop in temperature, together with drawing from a depth 105 ft. below the surface, brought amorphous matter from the bottom, and increased the manganese in the reservoir outfall to 0.27 p.p.m. As at Wanaque, the control of chlorination was made difficult.

The addition of lime at Wanaque accelerated the oxidation and precipitation of manganese, and soon after being placed in service, a coating, which contained a high percentage of manganese, formed on the insides of the steel aqueduct lines. Had the aqueduct been of new concrete, the highly alkaline surfaces would have collected more manganese which, later, would have sloughed away and impaired the appearance of the water. This was the case at New York and also at Providence.

Providence is supplied from the Scituate Reservoir, whose catchment area, from a geological standpoint, is not unlike that of the Wanaque and Croton areas. The impounded water was filtered to remove the color, sulphate of alumina being used as a coagulant, and the optimal pH value

for coagulation was about 6.0, which is too low for the removal of manganese by this kind of treatment. Lime was added to prevent corrosion. From the Scituate Reservoir, the water is delivered through seven miles of concrete-lined tunnel and pipe lines to the old distribution system in the city. The new distribution system also contains seven miles of large concrete pipe.

As stated by Craig, Bean and Sawyer,[8] treatment with sulphate of alumina was successfully used for this water for five years, but beginning in October, 1929, and following increased doses of lime in the filter effluent, complaints began to arise because of "red water," and "black sediments" were stirred up from the pipes wherever the velocities were unusual. These were the first evidences of manganese in the distribution system, and led to an investigation which on August 8, 1931 ended in using ferrous sulphate and lime in place of sulphate of alumina to treat the water. It was found that raw water drawn from the bottom of the Scituate Reservoir contained as much as 3.1 p.p.m. of iron and 0.75 p.p.m. of manganese. While the liming of the filter effluent to prevent corrosion had not raised the pH value high enough to precipitate the manganese, contact with the alkaline concrete surfaces of the aqueducts and the action of the manganese bacteria had undoubtedly done so. Eventually the accumulations so formed sloughed off and ultimately reached the consumer. This condition has been entirely remedied, and at present the delivered water contains no manganese.

Manganese has also occurred in troublesome amounts in the Swannanoa River supplying a bleachery at Biltmore, N. C. Just above the intake of the plant was a new dam forming a reservoir of considerable size. Soon after starting the plant, there was enough manganese in the filtered water (about 0.3 p.p.m.) to stain the walls of the filtered-water basin and collect on the rubber squeeze-rolls in the washing machines. Manganese in the raw water has decreased since the plant was first started four years ago, indicating unquestionably that its presence was due to the solution of the metal from the rich organic soil in the bottom of the new reservoir, especially during periods of stagnation.

Nearby is the plant of the American Enka Corporation, a rayon plant supplied from an artificial lake, having a capacity of 300 m.g., in the lower Hominy Valley, which drains the lower slopes of Mt. Pisca near Asheville. Conditions regarding this supply have been described by Paul W. Frisk.[9] The reservoir is supplied from Hominy Creek by pumping, and the filter plant of the Corporation is supplied, in turn, from the reservoir, which has a small catchment area of its own. As at Providence and elsewhere, manganese was not removed by ordinary chemical treatment. Analyses of the Lake Enka water at the surface and at various depths showed an increase in turbidity, iron, manganese and carbon dioxide, also a corresponding decrease in oxygen proportional to the increase in depth. Whenever the lower strata were disturbed, as by changes in temperature, excessive amounts of manganese appeared. The water of the lake contained 0.7 p.p.m.

of manganese, while the water entering contained little or none, indicating, as elsewhere, the solution of this element as the result of the decomposition of organic vegetable matter.

The remedy being applied to this reservoir is a continuous drawing of water from the bottom during the summer and early fall in order to prevent the accumulation of manganese with the hope that as the reservoir ages, conditions will improve. The water is chemically treated and filtered.

Summary.

The experiences of the authors and others show quite clearly that the chief cause of the presence of manganese in the water of impounded reservoirs is the decomposition of organic vegetable matter in the bottoms thereof, resulting in an acid condition and in the consequent solution of manganese. This action is particularly noticeable when hydrogen sulphide is present. Whenever a reservoir overturns, especially in the fall, manganese is distributed through the whole body of the reservoir, and because it oxidizes but slowly, is carried through the distributing aqueducts and pipes.

Most natural waters containing manganese have too low a pH value for the rapid oxidation of the manganese. Consequently, relatively little of the manganese, compared with the iron, is oxidized and re-precipitated in the reservoir. As long as the pH value of the water is low, manganese will oxidize and precipitate but slowly, but contact with highly alkaline concrete surfaces, or with manganese bacteria lining the inside of the pipe work, will result in precipitation with subsequent "black water," and ultimately complaints will arise.

Because the ortho-tolidine test may be seriously affected by manganese, it should be controlled or supplemented by other tests.

The presence of manganese in surface water is a nuisance, one most apt to occur in new reservoirs. As the reservoir ages, the amounts of manganese dissolved from the bottom soil decrease. In practice, the manganese nuisance can be greatly minimized by drawing off the bottom water during periods of stagnation.

In this paper we are indebted to the North Jersey District Water Supply commission and particularly to Mr. J. William Griffin, Superintendent of Operations, for permission to use the information regarding the Wanaque Reservoir.

REFERENCES.

[1] Forsberg. *Jour. A.W.W.A.*, *15*, 706, (1926).
[2] Forman. *Jour. A.W.W.A.*, *21*, 1212, (1929).
[3] Weston. *Trans. A.S.C.E.*, *64*, 175, (1911).
[4] Arthur H. Pratt. Jour. N.E.W.W.A., *44*, 387, (1930).
[5] von Wolzogen Kühr. *Jour. A.W.W.A.*, *18*, 1, (1927).
[6] D. D. Jackson, Trans. Am. Micro. Soc., *23*, 31, (1901).
[7] Frank E. Hale. *W.W. Eng.*, *79*, 970. (1896.)
[8] E. C. Craig, E. L. Bean and R. W. Sawyer. *Jour. A.W.W.A.*, *24*, 1762, (1932).
[9] Paul W. Friak. *Jour. A.W.W.A.*, *24*, 425, (1932).

RECONSTRUCTION OF THE LEBANON, N.H., WATER WORKS.

BY RICHARD S. HOLMGREN.*

[*Read December 14, 1932.*]

The water supply of Lebanon, N. H. is somewhat unique among those of New England towns in that water is drawn directly from the river within the town limits and after treatment by rapid sand-filters is pumped to the distribution reservoir by a hydraulic turbine-driven pump, utilizing water from the river for power. By this means an economical water supply has been developed with a minimum yearly operating cost. It is the purpose of this paper to describe this plant in the hope that it may suggest to other small towns a means of obtaining a low-cost supply. New England abounds with many small streams suitable for this purpose and capable of giving to the community an economical source of power for pumping, as well as a source of supply.

Lebanon, with a population of about 6 000, is located on the Mascoma River about three miles above its confluence with the Connecticut River at White River Junction, Vt. It is an industrial town with woolen and wood-working mills and several machine shops. The river has considerable fall throughout its length and is utilized extensively for power. Storage reservoirs have been built in the upper reaches of the river to be drawn on during the dry summer months, when the natural flow is low. About 900 mill. cu. ft. of storage capacity is now available above Lebanon, where the drainage area is about 150 sq. miles,— sufficient to insure a low flow of about 100 cu. ft. per second at Lebanon instead of about 40 cu. ft. per second — the approximate natural flow with no storage.

These reservoirs have been built and are operated by the Mascoma River Improvement Association, the membership of which consists of the various power users along the river, including the Lebanon Center Fire Precinct which supplies water to the town. In addition to the construction of reservoirs, the Association has general oversight and control of the river in order to keep its water as unpolluted as possible, for industrial as well as water-supply purposes.

The water-supply intake is located near the upper edge of the town, about three and one half miles below the outlet of Mascoma Lake. The town is fortunate in having this lake, with a surface area of about 1 000 acres, situated such a short distance upstream and on the main river, between it and the town of Enfield, another mill town. This provides a detention period for the stream flow and consequent clarification and purifying of the water during its passage through the lake, particularly at times of low flows.

There are a few houses bordering the river between the lake and the

*Principal Assistant Engineer with H. K. Barrows, Consulting Engineer, Boston, Mass.

plant, also an excelsior manufacturing plant using the water for power, but the water reaches the plant normally in good condition for treatment. There are times, however, during a heavy summer shower and during thaws in early spring, when the water becomes extremely poor in quality. It is the usual practice to shut down the plant for a short spell during the worst of these periods, and because of a large distribution-reservoir capacity this is possible without danger to the town.

Original Plant. The plant, as it existed before reconstruction consisted of a low timber-crib dam crossing the river and diverting the water into a canal through headgates located at one end of the dam. This canal, with an average width of about 30 ft., a depth of 5 ft., and a total length of about 500 ft., carried the water to the pumping station, a single-story brick structure with heavy granite substructure. The substructure was divided into two wheel pits, the one directly in front of the canal being for the original installation,— a water wheel and a Three-way pump with a capacity of 1 m.g.d. installed in 1889. The second pit was added on the north end at a later date and contains a water wheel driving a 1½-m.g.d. Laidlaw Dunn Gordon Duplex pump for active use, the old unit being retained for a standby. In 1921, a 650-g.p.m. Dayton Dowd electrically-driven centrifugal standby unit was installed in a nearby building, and the old Three-way pump was dismantled. In 1925, a second-hand Platt Iron Works Triplex pump with slightly higher capacity was purchased to replace the old Laidlaw pump, which was junked. Much trouble was experienced with this pump, constant breakage making the upkeep charges high.

The old filter plant consisted of 2 circular Jewell rapid sand-filters with a combined capacity of 1 m.g.d. located in an adjacent building. Water was drawn from the canal a short distance above the water-wheel intake and led directly to the filters. No coagulation basin was provided though a coagulant in the form of Alum was added in solution just back of the intake. After filtration, the water was discharged into a large clear-water well located some distance north of the plant. Pump suctions were carried from this clear-water well.

Failure of Plant. Early in October 1928 the headgates to the canal had been closed and the water drawn out in order to clean the canal of debris which had floated and rolled down stream during the summer and fall and had been diverted by the dam into the canal. After several days of cleaning the canal was filled with water and the pump started. Hardly had the water-turbine gates been opened when a loud crash occurred and the wheel jammed and stopped. The operator closed the water wheel gates and went outside to find water coming out of the north bank, gradually washing away the soil. The operator, fortunately, had the foresight to go to the head of the canal and close the gates, thus saving the station from being completely undermined. Upon examination it was found that water had forced its way along the north side of the station and underneath the wheel pit, causing the wheel foundation to settle several inches. This dis-

turbed the alignment of the water-wheel to such an extent that the gears jammed and operation was impossible. Water had also washed out material underneath the underpinning and partly filled the tailrace.

On inspection, it was found that no water could be held in the canal without rebuilding the foundations and cut-off walls of the station. As the water-supply intake leading to the filters was located just above the pumping station and below the canal headgates, operation of the filters was also impossible. It was therefore decided to extend the intake pipe upstream about 50 ft. to a temporary intake constructed as part of a cofferdam across the canal above the station. Some difficulty occurred in making the cofferdam tight at the south end, due to the presence of ledge close to the surface, but this difficulty was finally overcome. As the canal had been shut down for several days previous to the washout and several days were required to construct the cofferdam and new intake, the distribution reservoir was approaching a dangerously low level when water was finally run through the filters to the clear well. Only a few hours' supply remained in the service reservoir when the electric pump was finally started. It was quite evident that the large distribution-reservoir capacity had saved the town from a water shortage and proved the wisdom of previous water commissioners.

Reconstruction. Before any work was done toward rebuilding the station, a thorough study was made to determine the relative costs of different sources of supply. Several possible upland gravity supplies were investigated, but the first cost and yearly cost were considerably more than that necessary to rebuild the hydraulic-turbine pumping-unit and filters and put the plant in first-class shape. A study was also made considering abandoning the pumping station and using electric power for all pumping, but the yearly cost figured considerably higher than for power obtained with the reconstructed pumping station. It was therefore decided to rebuild the pumping station, and work was started on December 1, 1928, and carried on through the cold weather. (See Fig. 1.)

The final pumping-unit layout consisted of a new Rodney Hunt vertical water turbine located in the original wheel pit, having a capacity of 150 h.p. at 11-ft. head with a speed of 155 r.p.m. The maximum guaranteed efficiency of this wheel was 90 per cent. Connection was made to a Goulds horizontal 3-stage centrifugal pump running at 1 200 r.p.m., through a Philadelphia Gear Co. speed increaser having cut-steel herringbone and cut-steel spiral bevel gears with complete ballbearing mounting in a heavy cast-iron housing. This gear case has a ratio of speed increase of approximately 8 to 1 and changed the line of drive from vertical to horizontal. It came completely assembled ready to set it in place, connect and fill it with oil.

The centrifugal pump has a capacity of 1 100 g.p.m. when pumping against a total head of 285 ft., with a guaranteed efficiency of about 74 per cent. The tailrace, headworks and racks were enlarged and redesigned for

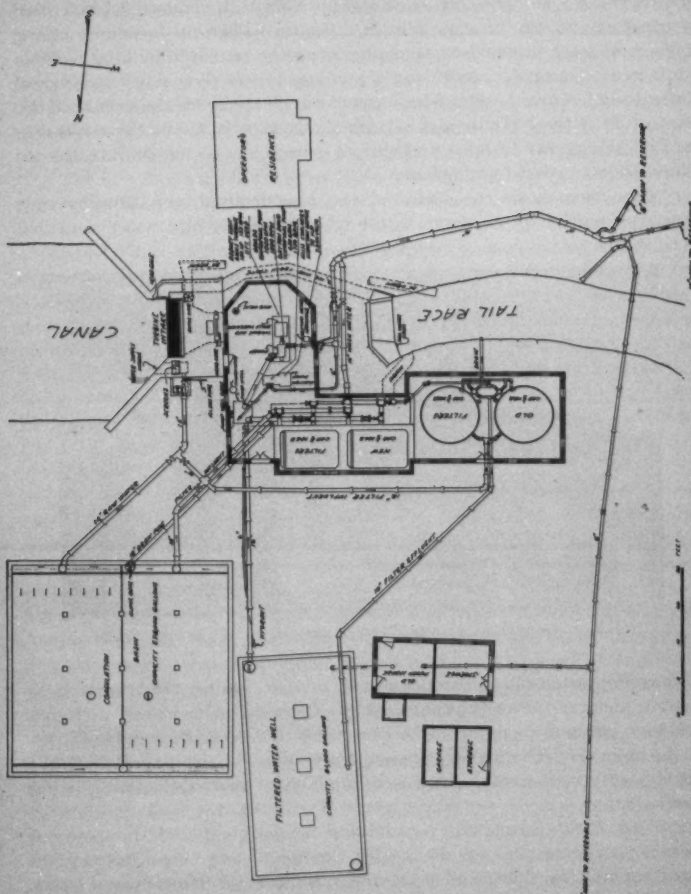


FIG. 1. — GENERAL PLAN OF RECONSTRUCTED WATER WORKS, LEBANON, N. H.

better hydraulic efficiency when handling the larger quantities of water used by the new wheel. The draft tube and pit were built somewhat lower than is necessary for the present tailrace, in order to permit deepening of the tailrace at some future date, thus increasing the head and making possible an increased pumpage, particularly at times of low river flow. Additional interesting features of the hydraulic structure were an ice sluice around the south side of the building and a trash sluice directly above the racks into which rakings are dropped and sluiced away by a water jet into the ice chute and so passed by the plant.

For convenience in operation, the electrically-driven standby unit was later moved to the main station when the main unit was completed.

FIG. 2. — ADDITIONAL FILTER UNITS, LEBANON, N. H., WATER WORKS.

This concentrated all pumping units and made possible the recording by Venturi meter of the amount pumped by either unit. Previously no record had been made of the pumpage by electricity. It also made possible the use of the main chlorine unit for applying chlorine to either pump. Previously an old, rather uncertain chlorinator had been used when pumping by electricity.

Filter Plant Reconstruction. During reconstruction of the pumping station it was decided by the Water Commissioners to reconstruct the rapid sand-filters. This had been under consideration for several years. It was also decided to build a coagulation basin, which was badly needed. By enlarging a small building connecting the pumping station with the filter house, it was possible to place the new filters there without interfering with the operation of the old filters. The old filters had been in use for many years and were doing fairly good work, but frequent breakage of parts was making the upkeep rather high. It was also difficult to control the flow through them as they were not equipped with rate controllers.

This resulted in a rather poor effluent, particularly directly after washing. Some difficulty had also been experienced in supplying the demand in a period of 10 hours, making overtime work necessary. As it was desired to employ only one work shift, two new filters were decided upon, each having a capacity of ½ m.g.d. It was planned to use these continuously, reserving the old filters as emergency standbys and possibly using them at a very low rate of flow. Any future enlargement of the plant would be made by removing these old filters and substituting new filters similar to those installed.

The new filters are of the usual type (see Fig. 2), each being a concrete tank 17 ft. long by 15 ft. wide having an effective sand area of about 180 sq. ft., equal to slightly under 1 sq. ft. of sand area for each 2 g.p.m. of capacity. The collector system in the bottom of each tank consists of a central 10-in. pipe manifold with 2-in. C. I. materials. A row of ¼-in. holes was drilled on each side of the laterals 6-in. on centers at the lower quarter points. Holes in the opposite sides of each pipe were staggered to obtain as even a distribution of holes as possible. The under-drainage system is surrounded and covered by 18 in. of screened gravel graded and placed in layers varying from 2½-in. at the bottom to that passing a No. 12 screen at the top. Above this gravel bed is a 30-in. layer of Plum Island sand having an effective size of about 0.4 m.m.

Rate of flow through each filter is controlled by a Builders Iron Foundry 6-in. rate controller with scale beams, and indicating rate-of-flow and loss-of-head meters located on the operating floor directly above the pipe gallery. The effluent from the filters flows by gravity to a 5 ft. by 6 ft. pump well located at one end of the pumping station. This well is also connected to the bottom of the filtered-water basin, so that a free interchange of water occurs between pump-section well and basin. Construction of this suction well was found advisable, even though it meant rather deep work in water-bearing gravel, because the clear water basin, which had been previously built, was situated at some distance from the station and its use would have required long pump-suction piping.

Beds are washed by introducing water into the under-drainage system directly from the force main, after shutting off influent and effluent lines and opening a valve to a drain located under the pipe gallery. Wash water is introduced to the filters at a rate of about 2 700 g.p.m., equal to a vertical rise of about 24 in. per minute. Waste water is collected in two steel-plate troughs, located over the sand bed, which collect the water from the surface and discharge it into the well beyond the baffle wall, whence it is discharged into the drain and tailrace. Washing is continued until the run-off is clear; a period of about five minutes.

In addition to the filters, a coagulation basin was constructed, 57 ft. by 57 ft. in plan, approximately 11 ft. deep, with a capacity of 270 000 gal., and divided centrally by a partition wall. Raw water after treatment with alum is introduced into a trough at one end of the east half and passes

through small holes in the bottom of this trough. In this way it is fairly evenly distributed across the entire width. The water then travels to the opposite end of this half, where it passes over a weir into a similar trough, which in turn carries it to the other half of the tank and passes it through another series of holes in the bottom of this trough, again evenly discharging it into this half of the tank. After moving through the complete length of the tank the water is skimmed over an adjustable weir and is then conducted by C. I. pipes to the filters.

The floor of the coagulation basin slopes towards the south, leading to a trough and drain pipe for cleaning out deposits in the bottom. A hydrant with full pressure is located near the basin so that when the basin is cleaned a hose is connected to the hydrant and, the nozzle is lowered through a manhole. By starting at the upper end all deposits are washed out. A pipe leads to the drain under the floor of filter pipe gallery and is there carried into the tailrace of pumping unit. The basin is somewhat larger than necessary for the present filters, but upon the construction of an additional filter unit in the future, the capacity of the entire plant will be balanced at 1.5 m.g.d., at which rate the detention period will be about four hours.

A suitable cross connection is placed in the piping so that the basin can be by-passed and raw water fed directly to the filters in case of an emergency.

Operation. Since completion of the plant in October, 1929, it has given the town a continuous and abundant supply of pure water. Pumps are normally started at seven each morning and continue, with an hour's shutdown at noon, until such time as the distribution reservoir is full; no pumping is done on Sundays except when the demand is so great that the reservoir is not filled by Saturday evening.

Filters are started at the same time and run in such a way as to leave the filtered-water well full at the end of the day. Alum is added to the water, at the intake to the supply system, by a dry feed machine, at a rate of 1½ to 2 grains per gallon. Chlorine is added to the filtered water as it passes through the suction of the pump at a rate of about 0.17 p.p.m. Tests for residual chlorine are frequently made, and proper control of the amount of chlorine applied is determined thereby.

Filters are washed when the loss of head through the filters becomes about nine feet. Washing continues for about 5 minutes. Filter runs vary from twelve to fifteen hours between washings.

Water Analyses. Samples of water from the river, the filter effluents and a faucet in town are sent monthly to the State Board of Health at Concord, N. H., for analysis. Tests for *B. coli* are made on all samples and though a positive result is invariably shown on the raw river water, the results for the other samples are always negative. No general analyses of the raw water are available, but the results of the analyses made on the faucet water range as follows:

Turbidity	very slight
Sediment	very slight
Odor	very faint, earthy
Color — p.p.m.	0.5
Ammonia Nitrogen — p.p.m.	0.006 – 0.026
Albuminoid Nitrogen — p.p.m.	0.024 – 0.048
Nitrate Nitrogen — p.p.m.	0.000 – 0.03
Nitrite Nitrogen — p.p.m.	0.000
Chlorides	0.9 – 1.3
Hardness	15.0 –32.0

Costs. The total cost of all the construction work done during 1928–9 was about $58 000 divided as follows:

PUMPING STATION.		
Reconstructing intake wheel pit and tailrace	$14 000	
Turbine, gates, gears and pump installed	7 000	
		$21 000
FILTRATION PLANT.		
Filter plant and coagulation basin, exclusive of piping	$22 000	
Filter piping, valves and controls	5 000	
Connecting piping and valves (installed by Water Department)	3 000	
		$30 000
Engineering		7 000
Total		$58 000

The cost of reconstructing the pumping plant not including the treatment works, was about $23 000; equal to about $150 per h.p. of installed water-power capacity. The yearly cost of pumping alone is about as follows:

FIXED CHARGES.		
Interest @ 4½%	= $1 040	
Depreciation @ 3%	= 690	
say		$1 800
OPERATING EXPENSES.		
Labor	$300	
Supplies	100	
		400
		$2 200

Based on present use of 0.5 m.g.d. or about 180 m.g. per year, the cost of pumping, including all fixed charges, is about $12.20 per m.g. or for a 290-ft. head a total cost per m.g. raised one foot of about 4.2 cents, of which only about ¾ cent is operating cost.

Of the $35 000 spent for the remainder of the work, the cost chargeable to the filters, including all piping and building costs, was about $23 000, equal to a cost of $23 000 per m.g.d. capacity. This unit price is somewhat higher than ordinary as there are included some costs of the buildings and

suction well, as well as provision for future filters not directly chargeable to these units.

The coagulation basin with its adjacent piping cost about $12 000. As the capacity of the coagulation basin is 278 000 gallons, this is equivalent to a unit cost of about $43 per thousand gallons.

Concluding. Work on the pumping station reconstruction was carried on during the winter of 1928–29 and was handicapped at times by extreme cold weather. Later work on the filtration plant required extreme care and careful planning by the requirement of providing the town with a continuous supply of clean water during construction. The plant described was completed in October 1929 and since that time has been in continuous operation, furnishing the town of Lebanon with an excellent supply of water at low cost.

This work was carried out by the Lebanon Center Fire Precinct under the direction of a joint committee consisting of a special construction committee appointed by the Town and the Board of Water Commissioners. The Water Commissioners were Messrs. Manson, Plamondon and Hyde, Mr. Plamondon being Chairman of the Board and Superintendent from the beginning to April, 1929, and succeeded by Mr. Manson for the remainder of the work. The contractors on the work were the Ames Construction Co. of Somersworth, N. H. and the W. H. Trumbull Co. of Hanover, N. H. The work was done under the direction of H. K. Barrows of Boston as Consulting Engineer, with design and construction in charge of the speaker, A. H. Engborg acting as Resident Engineer.

THE HIGH SPEED DIESEL ENGINE AS A PRIME MOVER FOR PUMPING STATIONS.

BY ELLIOTT EARL.*

[*Read November 9, 1932.*]

Some Facts on the Origin of the Diesel Engine. At present there are a number of people, engineers and others, who deprecate the use of the term Diesel in connection with the modern type of compression ignition oil engine. Just how much this modern engine is the outgrowth of the work of Ackroyd-Stuart in England and Diesel in Germany is, it seems, a controversy. In a paper read in April 1931, D. R. Pye, Deputy Director of Scientific Research at the Air Ministry, England, gives us the following salient facts: "Herbert Ackroyd-Stuart was born in 1864, trained as an engineer in his father's works at Fenny Stratford. Between 1885 and 1890, the records show that he took out several patents for the improvement of the oil engine. In one taken out in conjunction with Mr. Binney of London, (No. 7146 of 1890) there is a quotation of what is now known as one of the essentials of the true Diesel cycle, that is, *the compression of pure air alone.*"

Rudolph Diesel was born in Paris, in 1858 of German parents. He was educated at Augsburg and Munich. His works training was with Gebrüder Sulzer in Winterthur, Switzerland. Dr. Diesel's first English patent was dated 1892, and his object was to produce, quoting the patent: ". . . Motive work by means of heated air . . . compressed to so high a degree, that by the expansion subsequent to the combustion, the air is cooled about to atmospheric temperature, and into this quantity of air, after its compression, fuel is gradually introduced. . . . At this compression the temperature becomes so high that the fuel employed is spontaneously ignited when it comes into contact with the compressed air. . . ."

It would appear from the above, that honors for the invention of the compression ignition oil engine were divided between Ackroyd-Stuart and Diesel. Sometime later Mr. Busch of St. Louis successfully negotiated the American license, and maintained it until 1914. During the war the Diesel was brought to a state of perfection that is now being recognized. It is about this type of engine that comments will be made.

There have been many developments in materials, engineering technique, and machine shop practice since Dr. Diesel first propounded his theory of what a heat engine should be. The result is that the modern so-called Diesel engine is something quite different from what Dr. Diesel was working for.

A Definition of the True Diesel Cycle. All of us are familiar with the Carnot cycle, which gives the highest attainable efficiency in heat engines.

*Consulting Engineer, Boston, Mass.

Briefly reviewed, the Carnot cycle, consists of four changes: (1) an isothermal compression of a gas, say air in this case; (2) an adiabatic compression; (3) an isothermal expansion; (4) an adiabatic expansion. The proportions of these changes are such that the air at the conclusion of the events will be at the same temperature and pressure as at the start of the cycle.

However desirable this cycle may be, it seems that it is quite unattainable, for by definition of an isothermal change we know that the heat gen-

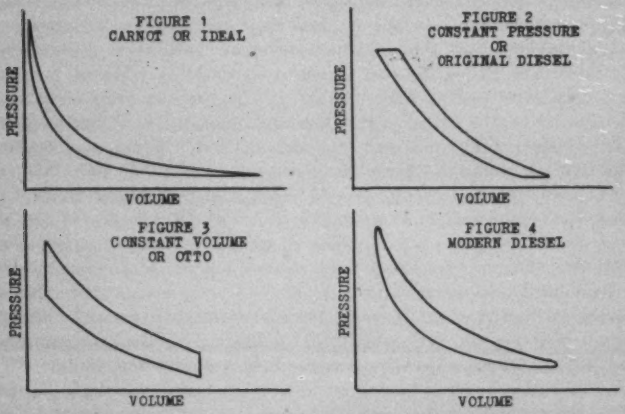


FIG. 1. — EVOLUTION OF THE MODERN THEORETICAL DIESEL CYCLE.

erated during change (1) must be conducted away as fast as it is generated; then during the second change the conditions must be reversed, *i.e.*, no heat must be carried away or taken from external sources. Similarly in the third and fourth phases of the cycle the conditions are opposed. To meet these conditions, the cylinder walls would have to be perfect conductors of heat in the first, and perfect non-conductors in the next phase. Because of this impossibility we have had to make compromises. The engine arrived at after these important alterations in cyclic events had gone far from the original Carnot idea and Dr. Diesel's conception of it; practically the only feature remaining being the spontaneous ignition of the sprayed fuel.

Dr. Diesel realized these limitations in his first designs; in his attempt to meet them, he made a compromise, which was, *combustion at constant pressure*. This defines what is now known as the true Diesel cycle.

If the fuel injection is commenced just before top center, the curve obtained is approximately flat at the top (constant pressure), but as the

moment of beginning the injection is advanced, the curve can be made sharper and sharper, with corresponding higher maximum pressures, until practically constant volume is attained. In the opinion of Dr. Langer, an eminent German internal combustion engineer, there is no merit in calling the constant pressure cycle a true Diesel cycle. It should be abandoned in favor of the Otto, (constant volume) cycle, or the combination, which is a little of both and is the present cycle. (See Fig. 1)

The Efficiency of the Diesel Engine. Going back to Carnot again. In his imaginary engine, he shows that when certain conditions of rejection and reception of heat of the working substance are fulfilled, the highest possible efficiency which any imagined engine can have is expressed by the fraction:

$$\frac{T_1 - T_2}{T_1}$$

where T_1 is the highest temperature encountered by the working substance and T_2 the lowest. As has already been shown, though the conditions for the ideal Carnot cycle engine can never be fulfilled in practice, and actual efficiencies are well below that of the Carnot engine for the same highest and lowest temperatures, the importance of a wide temperature range has been fully confirmed by practice.

If we compare then, the ranges of temperature available in the internal combustion cycle as it is today, with that in the most modern steam turbine plants, using a high vacuum condenser, we have the following:

Steam Plant	*Internal Combustion Engine*
$T_1 = 743°$ Absolute	$T_1 = 3073°$ Absolute
$T_2 = 313°$ Absolute	$T_2 = 773°$ Absolute

The corresponding Carnot efficiencies are:

For the internal combustion engine	75 per cent.
For the steam plant	58 per cent.

The steam-plant efficiency applies to the turbine alone. From even the most efficient express type boiler with superheaters, economizers, etc., not more than 85 per cent. of the fuel heat reaches the turbine. The overall efficiency of the plant will be the product of the efficiencies for the turbine and the boiler taken separately; one is not sufficient. The highest brake thermal efficiency achieved in practice today with a turbine has been 33.8 per cent. An internal combustion engine working at its full load output has attained 40 per cent. As pointed previously, it is the overall efficiency of the steam plant which must be compared with the brake thermal efficiency of the internal combustion engine, namely $33.8 \times 0.85 = 28.7$ per cent. as against 40 per cent. for the internal combustion engine. The reason for this difference, obviously, is that in the internal combustion engine the whole

process of power production is completed within the boundaries of the engine itself, from the chemically stored energy of the fuel to the crank effort on the shaft.

The trend in recent Diesel engine design has been towards increased rotative speeds with attendant increased mean effective pressures. The result is that the modern Diesel cycle is approaching the constant volume or Otto cycle. There are various reasons for this. It is enough to say that the general shape of the Otto cycle has more of the characteristics of the ideal Carnot cycle than did the original constant pressure Diesel cycle. The step by step departure from the ideal, to meet practical limitations, suggests that the engine arrived at has a thermal efficiency which is as high as is attainable under existing conditions; but it also suggests that if friction can be reduced and higher pressures safely used, a partial return over some of the steps which took us away from the Carnot cycle and Diesel's original plan may be possible, resulting in still higher efficiency. It is a rather striking fact that numerous Diesel engines made over twenty years ago had, and still have, a fuel consumption practically equal to that prevailing today.

The efficiency of a steam unit is usually expressed in foot pounds of work done per thousand pounds of steam used. This can be expressed in foot pounds of work done per million B.t.u.

An article by Couch in this JOURNAL stated that the Diesel unit discussed had an actual duty of 345 400 000 ft.-lb. per 100 pounds of fuel consumed. Assuming that the fuel was oil which had a fuel heat content of 18 000 B.t.u. per pound, this plant has an efficiency of 196 000 000 ft.-lb. per million B.t.u. In a paper by R. L. Baldwin published in *Water Works Engineering*, January 15, 1930 a complete picture is given of the Diesel engine powered pumping station. Some interesting examples are cited. Baldwin states that most average steam plants will have a thermal efficiency of from 10 to 15 per cent. and that the largest will not exceed 28 per cent., with which I agree. He goes on to state that the average Diesel engine thermal efficiency is about 32 per cent. It is my opinion that this is low.

Because of economy of space and material, it seems to me that any slow speed prime mover must ever give way to its high speed offspring. Consider the development of the steam engine. It is thought that the internal combustion engine is bound to proceed in the same general way.

There are some pumping stations in this vicinity which will so awe the spectator with their romance, magnificence, stately engines, polished mahogany and brass, as to make him completely forget such an important item as economy of operation. In performing their function of pumping water, these plants have slow unhurried strokes, moving with a ponderous dignity. A tribute to the engineering genius of Watt. The contention is that pumping water is basically a function of power production. If such is the case, it is further contended that one of the most economical methods of doing so is through the utilization of a piece of machinery that looks no

more imposing than an oil can and has about as much romance and dignity.

Any conservative engineer knows that what has been told so far is not a complete picture of the case. A low fuel consumption per shaft horsepower, of course, is one of the most important factors in the economics of power production. Reliability, the first cost of the plant, maintenance, labor, all have to be considered.

In the matter of reliability: in aviation the first function of an airplane is to maintain itself in flight, as long as the engine runs the plane will stay in flight, all other parts functioning in an equal manner. Do I need to say more than that the winner of the Collier aviation trophy in 1931 was the high speed Diesel engine.

In the matter of first costs: Baldwin states that the first cost of a Diesel engine powered plant in most cases would be as high and in many cases higher than the other types. (Steam and electric.) I agree. However, in this year of distress one manufacturer is known to have quoted a plant at $55.00 per installed B.H.P., which is thought will compare favorably with either steam or electricity.

In the matter of maintenance: the 1931 Cost Report of the American Society of Mechanical Engineers Oil Engine Power Sub-committee reveals that for 119 plants containing 330 engines the average cost of engine repairs was 0.52 mills per net kilowatt hour of power generated. The belief that oil engine repairs were less than steam has been held for years by engineers familiar with Diesels. This report clearly sustains their position.

In the matter of labor: Kates, in a paper before the Fourth Oil Power Conference of the American Society of Mechanical Engineers, June 1930, entitled "Joint Use of Diesel and Steam Engines to Balance Heat and Power," states the following: "The intelligent steam operating engineer has no difficulty in learning to handle Diesel engines." Obviously any average cost will be in favor of the Diesel, since no license is required to operate internal combustion engines.

Cost Comparisons. Up to this point the subject has been theoretical and of a thermal nature. To give it a practical meaning, costs in dollars will now be considered rather than British thermal units. It is emphasized that this is a purely preliminary analysis which has the single purpose of demonstrating the advisability of carefully detailed investigations in considering the problem of Diesel powered pumping stations.

Twenty years ago when the cost of labor, fuel and supplies was relatively reasonable, a certain new pumping station of the steam, crank and flywheel type, had a possible daily capacity of 145 m.g. It had a daily average duty of 61 740 000 gal. for an average lift of approximately 84 ft. In terms of equivalent power costs, the plant was operating for a cost of 8.3 mills per equivalent k.w.h. Fixed charges are not considered. In 1920 the plant was unchanged, it was pumping for an equivalent of 20.8 mills per equivalent k.w.h. The plant, unchanged is operating today. In

1931, the daily average was 54 500 000 gal. for an average lift of about 84 ft. The cost was approximately 16.4 mills per equivalent k.w.h.

A comparison with a hypothetical electric plant and an actual Diesel plant is interesting.

If electric power could be purchased for 11 mills and this source of power utilized satisfactorily, there would be a possible saving of 5.4 mills per equivalent k.w.h.

There is in operation today a Diesel engine power plant described by Anderson, in the October, 1932 issue of *Diesel Power*, which has a comparable power rating with the pumping plant previously described. For the last year the plant has produced power for 6.41 mills per equivalent k.w.h. Comparing this cost with some of those reported in the American Society of Mechanical Engineers Oil Engine Power Cost Report for 1931, it is low but not exceptional. In a pumping station it is advisable to connect the Diesel directly to the pumps to obviate conversion losses. The saving per equivalent k.w.h. is then, 10 mills or 1 cent with the Diesel plant.

Reducing this saving to dollars annually, and assuming that the present steam plant costs in the neighborhood of $86 300 annually to operate, there would be a saving of $28 600 per annum if purchased power were utilized as prime mover equipment. With a properly engineered Diesel installation, the indicated savings are of the order of $52 500 per annum. Going into the matter a little further and assuming that the Diesel plant had a burden of approximately 4.6 mills per equivalent k.w., it could still show a saving equal to that of the electric installation. Another way of looking at this saving is in terms of the amount available for plant retirement. There will be approximately $24 000 available per year over the purchased power method.

Some Advantages of High Speed Diesels. A consideration of the burden of pumping stations brings us back to the subject of the discussion, high speed Diesel engines. High speed and small size are relative terms of course. By high speed is meant any engine with a piston speed of 1 200 feet per minute or more, and by small size is meant cylinders having diameters of eight inches or less. There are very good reasons why we should keep down the sizes and increase the speed of our engines. By keeping down the sizes we reduce first cost considerably, to wit, we do not have to buy a lot of unnecessary cast iron. We reduce burden charges, since unnecessary speed increases by gear sets are eliminated and first cost is lowered. Above all, we keep clear of heat stresses.

The obvious essential advantage of light weight is ease of transportation. The other advantage is low cost of manufacture. The gasoline engine today costs from $5 to $15 per horsepower according to the manufacturer's production ability. There is no reason why the high speed Diesel should not ultimately come into this range. Certainly the workmanship and material required are no better than necessary in a reputable gasoline engine. It is believed that we are on the eve of seeing light high speed Diesel

engines turned out in quantity production. One engine manufacturer is known who has an oil engine which will satisfactorily compete with his best gasoline product. If such is the case, we should revise our ideas as to prime mover requirements. In the first place, the efficiency of a Diesel engine is independent of size. As has been brought out, the Diesel engines chief competitor is the super-power steam plant, which is efficient only in very large units. Where Diesels are concerned it is contended that we should employ a number of units of the smaller size: because (1), we gain in efficiency of operation since the plant and each individual unit can be operated at the most economical load factor; (2) first cost is lowered; (3) most important, there is increased reliability of operation.

The arguments in opposition to this reasoning seem to lack weight. One is asked to look with ridicule at a power plant or pumping station having a series of Diesel engines installed with anywhere from 4 to 40 pistons and with their accompanying 8 to 80 valves, all of which with their accompanying lubrication systems, cooling systems, and fuel injection systems must function properly at all times. We are then asked to look with comfort and assurance at a pumping station or power plant, which has a turbine or an electric motor doing the same work. At first glance we may say the comparison is appalling. Let us analyze it a little more closely.

We know that with a multiplicity of Diesel units, one, two, three, or four, may go wrong at the same time, yet the rest of the installation will carry the load temporarily, while the faulty units are being repaired, assuming of course that the plant has been properly engineered at the outset.

Let us consider the turbine as a single unit plant. Inside of the turbine's simple outside casing there are a thousand or more little blades, unseen but not to be forgotten. The failure of one of those blades will bring the turbine to a complete standstill, and if we are not fortunate enough to have provided for suitable standby equipment, our system is without a source of supply. Happily, the pictured occasion is fairly remote.

The electric motor does not have any blades, and from a casual analysis of its reliability, it could safely be said that there are few other pieces of machinery with as much inherent reliability. In using an electric motor unfortunately there must be an outside source of power. The station is then subjected to all of the inherent weaknesses of a steam or hydraulic power plant, and the possibility of transmission line failure in addition to the remote possibility of control equipment and motor failure. The electric motor with an outside source of power supply is not a good solution of the pumping station problem as one community found out last winter through transmission line failure.

Let us reconsider the Diesel engine as prime mover or standby equipment for the pumping station. The chance of two or three Diesel engines failing at the same time is beyond the bounds of probability and almost possibility. Who of us who is fortunate enough to own two motor cars has had them fail due to engine trouble at the same time? It is a matter of

record that the Diesel engine of today is as reliable as any gasoline engine ever built. There is a reason for this. When one eliminates the carburettor and ignition system from an internal combustion engine one has eliminated practically all of the sources of trouble. Such an engine is the Diesel.

It is my confirmed belief that the Diesel engine of the near future will be a small high speed unit of such a size that it can be turned out cheaply,

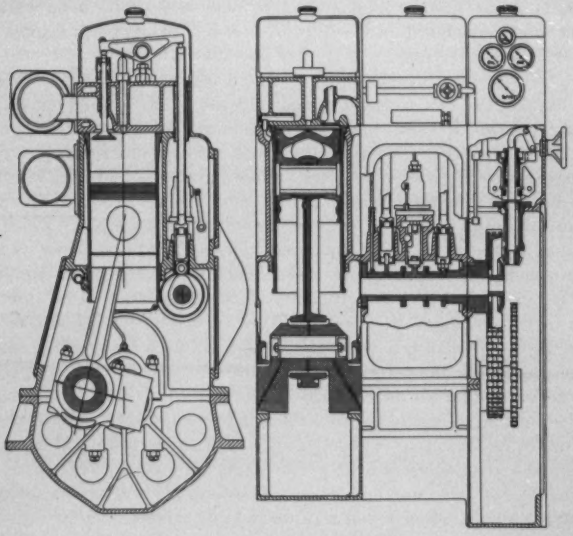


FIG. 2. — TRANSVERSE AND PARTIAL LONGITUDINAL CROSS-SECTION OF A MODERN HIGH-SPEED DIESEL ENGINE.
(Courtesy of Journal of the Society of Automotive Engineers.)

and that where large power concentrations are needed, we shall employ a battery of these units. Our method of handling them will then be much as we handle our trucks, and cars today.

The Problem of Combustion in Diesel Engines. Going on to a little more technical discussion of the Diesel, Ricardo, one of the most renowned authorities on internal combustion engines, describes the combustion process in the modern high speed Diesel engine, in the following unique manner:

Accompany me in imagination inside of the cylinder of a Diesel engine. Let us imagine ourselves seated comfortably on the top of the piston, at about the end of the compression stroke. We are in complete

darkness, the atmosphere is a trifle oppressive, for the shade temperature is well over 500° C — almost a dull red heat — and the density is such that the contents of the average sitting room would weigh about a ton; also it is very draughty, in fact the draught is such that in reality we should be blown off our perch and hurled about like autumn leaves in a gale. Suddenly, above our heads a valve is opened, a rainstorm of fuel oil begins to descend. I have called it a rainstorm, but the velocity of the droplets approaches much more nearly that of rifle bullets than of raindrops. For a while nothing startling happens, the rain continues to fall, the darkness remains intense. Then suddenly, away to our right perhaps, a brilliant gleam of light appears moving swiftly and purposefully; in an instant this is followed by myriad others all around us, some large and some small, until on all sides of us the space is filled with a merry blaze of moving lights; from time to time the smaller lights wink and go out while the larger ones develop fiery tails like comets; occasionally these strike the walls, but being surrounded by an envelope of burning vapor they merely bounce off like drops of water spilt on a red hot plate. Right overhead all is darkness, still, the rainstorm continues, and the heat becomes intense; and now we shall notice a change is taking place. Many of the smaller lights around us have gone out, but new ones are beginning to appear, more overhead, and to form themselves into definite streams shooting rapidly downwards or outwards from the direction of the injector nozzles. Looking around again we see that the lights are growing yellower; they no longer move in definite directions, but appear to be moving listlessly hither and thither; here and there they are crowding together in dense nebulae, and they are burning with a sickly smoky flame, half suffocated for the want of oxygen. Now we are attracted by the dazzle overhead, and looking up we see that what at first was a cold rain of fuel oil falling through utter darkness, has given place to a cascade of fire as from a rocket. For a little while this continues, then ceases abruptly as the fuel valve closes. Above and all around us are some lingering fire balls, now trailing long tails of sparks and smoke and wandering aimlessly in search of the last dregs of oxygen which will consume them and finally set their souls at rest. If so, well and good; if not some unromantic engineer outside will merely grumble that the exhaust is dirty and will set the fuel valve to close a trifle earlier. So ends the scene, or rather my conception of it, and I will ask you to realize that what has taken me nearly five minutes to describe may all be enacted in one five-hundredth of a second or less.

Such is the problem of combustion and fuel injection in Diesel engines.

The Fuel Injection System of Diesel Engines. The injection of fuel in a compression ignition engine has definitely tended towards the so-called solid injection system. It might be wise to define solid injection. It is that type of fuel injection system which does not utilize compressed air in the process of injecting the fuel. The English way of expressing it as "airless injection" is a little more rational.

Having noted that the trend is towards the so-called solid injection system, the first type, *i.e.*, the air injection systems will be dismissed without further discussion except to say that, for the injection of certain types of the heavier grades of fuel oil, air injection possesses advantages which are not offered by the solid injection system.

The lack of agreement between engineers regarding the method of introduction of the fuel has resulted in many different designs. There are two outstanding systems commercially available and manufactured by specialty manufacturers. One is called the Bosch system, manufactured by the United American Bosch Company in Springfield, Mass. The other is the Compur system, manufactured by Friedrich Deckel of Munich, Germany. Both are of the jerk pump type.

The types of systems in existence today may be conveniently divided into two groups: (1) the jerk pump system; (2) the common rail or constant pressure system. In addition to these there are many modifications, which include distributor types and miscellaneous types which are mixtures.

The parts that go to make up the conventional system are: the nozzle or injector, the fuel pump, the supply line, and accessories, the latter forming a very important part of the system. When one considers the size of the passages in the fuel injection system, it becomes evident that the introduction of dirt and water will cause considerable damage. This fault was common to early American plants. It has been completely overcome through the development of centrifuging and filtering equipment, the cost of which adds very little to the total first cost.

An injection system has five duties to perform in order to introduce the fuel into the combustion space in an economical and rational manner. It must clean and transport the fuel from the storage tanks to the combustion space. It must accurately meter the fuel under all load conditions. It must time the fuel injection period with reference to the position of the piston. It must properly atomize the fuel. It must properly disperse the fuel.

Cleaning the fuel is accomplished by centrifuges as previously noted. Transporting it from the tanks to the combustion space is usually accomplished by means of low-pressure pumps. If heavy fuels are to be used a consideration of the heating problem is important, since it has been shown that viscosity is a function of the temperature, and more lately that viscosity has a great deal to do with atomization.

The elimination of air in the fuel injection system is of the utmost importance. Fine wire mesh screens and filters will help in this connection. The last four points constitute design problems. In properly designed systems, of which the Bosch system is one of the best, no trouble may be expected. In passing it might be said that early American designs did not recognize the existence of these problems. The result is that some of the people who invested in the engines are sorry. Without any hesitation I consider these points the cause for the late adoption of the Diesel in this country.

Continuing with the discussion of the fuel injection system it should be insensible to the following factors: speed, load, fuel, clogging, errosion, and wear. Further consideration of the system suggests the following: simplicity of construction, ruggedness, freedom from the possibility of the

introduction of air. Air in the fuel injection system of a Diesel engine has the same deleterious effect on the engine as that which most of us have experienced in getting water into the carburettor of our automobile engine. Most of the trouble, if there is any with a Diesel engine, can be traced directly to the fuel injection system. Common sense calls for the operator knowing as much as he can about it.

Fuel for Diesel Engines. The question of what kind of fuel should be burned in a Diesel engine is a good one. It has caused considerable discussion of late. In a paper presented before the summer meeting of the Oil and Gas Power Section of the American Society of Mechanical Engineers, this year, Stalnaker goes into the question with Bunker C as a background. The representatives of the oil companies present took exception to what he said in no unmistakable manner. The upshot of the whole matter is this: with the proper engineering supervision, there is no reason why any Diesel will not burn any fuel designated, which contains the proper fuel heating value, and the proper physical properties, which in turn will permit of injection. There has been a lot of loose talk in the past about fuels for Diesel engines. Not the least of these has been the suggestion by the lay press and others that these engines are able to run on any kind of oil, even refuse oil from other machines. *That these engines will run on this type of fuel is a fact, but that such operation is advisable is another matter.* There is a committee in existence today, coöperative in nature, composed of some of the best minds in the leading engineering societies in whose province this matter falls, who are working on the problem of the standardization of Diesel fuel oil. Until the time that these standards are set it might be wise to abide by the decision of the manufacturer in regard to the matter of fuels. The net cost of operation will probably be slightly in favor of the lighter fuels.

Conclusion.

We are accustomed to telling ourselves that we live in an age of progress. All new ideas fight with old ones. All changes in organization, all modifications of practice mean that a new method either wholly or partially ousts an old. That is what progress means. If we really believe that we are progressing what is the use of our buying engines that last more than ten or fifteen years. Our fathers and forefathers were wont to boast that they built machinery to last a lifetime. In so doing they built mighty impressive monuments to themselves but mighty embarrassing heirlooms for their descendants. Today, much of our trouble is due to the sturdy vitality of machinery which has long since grown obsolete and paid for itself, yet will not die.

In re-capitulation, we have seen that a Diesel engine is theoretically and practically the most efficient type of prime mover made today. While it is not claimed that Diesel engines are going to be the universal panacea for that disease known as "costly power," it is claimed that they should be investigated as a means of lowering pumping costs, and as one of the most

reliable types of standby equipment. It is not claimed that the Diesel engine will guarantee an adequate supply indefinitely. What is claimed is that if a pumping station is powered with Diesel engines, assuming that it has been properly engineered, we shall have a more reliable source of water supply, for a somewhat lower net cost.

In the words of Ricardo: "Low first cost, light weight, little space requirements, ease of transport, reliability, these are our needs. Why not be content to recognize them leaving something for our descendants to use a free hand on with their better knowledge and altered conditions."

Discussion.

Arthur L. Shaw.* As experience with Diesel engines accumulates and confidence in their reliability grows, the frequency of occasions is increasing when the engineer must consider them in comparison with other prime movers. Mr. Earl's paper, therefore, is a timely exposition of the Diesel side of the argument. As he rightly states, however, there are two sides to the question and each problem must be considered on its individual merits.

Up to now the fixed charges due to high first cost of Diesel engines have often been the stumbling block in estimates comparing them with other types. At prevailing prices Diesels have usually appeared uneconomical in total annual cost except in those instances where practically continuous operation has made it possible to take full advantage of admitted operating economies as an offset to high fixed charges. If, as the author prophesies, the unit cost of the Diesel engine is cut materially below the figures that have prevailed in the past, the complexion of comparative estimates for a given undertaking may completely change. General conclusions based on current quotations, however, should be drawn with caution, until it shall be demonstrated that current prices represent a true downward trend and not temporary economic distress. Should unit costs drop into the price range of high-grade gasoline engines, Diesel engines will find many applications as standby and part-time units, where gasoline engines are now commonly employed. Such prices are, however, as yet but a prophecy.

The title of this paper refers to "high speed" Diesel engines. Undoubtedly manufacturers must look to high speed if the price per horsepower is to be materially lowered. Much of the author's argument, however, is predicated upon the results of experience with the more familiar and well-established low-speed Diesel engine. These data must be applied with caution to proposed high-speed installations until more is known of the ability of the high-speed oil engine to meet municipal pumping requirements which demand steady and reliable service for days, weeks and months. It is to be expected that the percentage for repairs and maintenance will be greater, and the years of life shorter, for the high-speed Diesel than for the slower machines which have supplied most of the existing information on this subject. Present experimental work with oil engines for

*Of Metcalf & Eddy, Engineers, Boston.

airplanes and automobiles is of interest and may eventually supply the needed data, although this type of service is not directly comparable with the steady continuous service required of most pumping units.

The author has mentioned the weakness of those installations which depend upon purchased power. To correct this weakness the electric-power companies can frequently be persuaded to furnish two or more sets of feeders to pumping stations where a considerable block of power is involved. One of these feeders can often be placed underground, thus minimizing the hazard of loss of power through transmission-line failure. Furthermore, in view of the desirable character of the load at many pumping stations, the power companies will sometimes go so far as to establish special rate schedules which will correspond closely with the cost of power from Diesel engines. Even the Diesel engine is dependent upon the delivery of oil at intervals from outside sources, although adequate storage facilities minimize the hazard from this agency.

There is much to be said for the author's argument against planning too far into the future in these days of rapid mechanical progress, and yet it must not be forgotten that some of the older plants which he styles monumental and ponderous are, after thirty, forty, or more years, still showing duties that only the best of the newer and shorter lived high-speed steam units can approach. If this experience is repeated in the oil engine industry, as the development tends toward lighter and higher speed units, the higher efficiency of the larger and slower machines will sometimes dictate their use instead of smaller faster engines.

If this should prove to be the case the author's future pumping station, containing numerous small units individually driven by high-speed Diesel engines, will sometimes find itself in competition with the more conventional plan involving the use of a relatively large slow-speed Diesel engine driven generator with a second similar unit in reserve, supplying power to a number of motor-driven pumping units. This arrangement provides similar flexibility of pumping capacity and would take advantage of the probable higher efficiency of the larger Diesel unit, which would in a measure offset the intervening generator and motor losses, and would have further advantages in those cases where there are isolated pumps outside of, but not far from the main station. This situation frequently occurs with ground water supplies.

The author, in effect, defines progress as "the defeat of old ideas by new ones." This definition gives too little credit to the past, upon the experiences of which real progress is founded. Perhaps it is New England conservatism which dictates that new developments should await thorough establishment before being generally applied to anything so vital as the pumping of water for public supplies. Much progress has been made with the Diesel engine, and in a conventional form it is today acceptable where circumstances show it to be economical. It is not improbable that the high-speed Diesel engine will ultimately be developed to have similar application to water-works pumping problems.

RELATION OF GEOLOGY TO THE GROUNDWATER SUPPLIES OF NEW ENGLAND.

BY IRVING B. CROSBY.*

[*Read January 11, 1933.*]

[The superior figures refer to references which will be found at the end of this paper.]

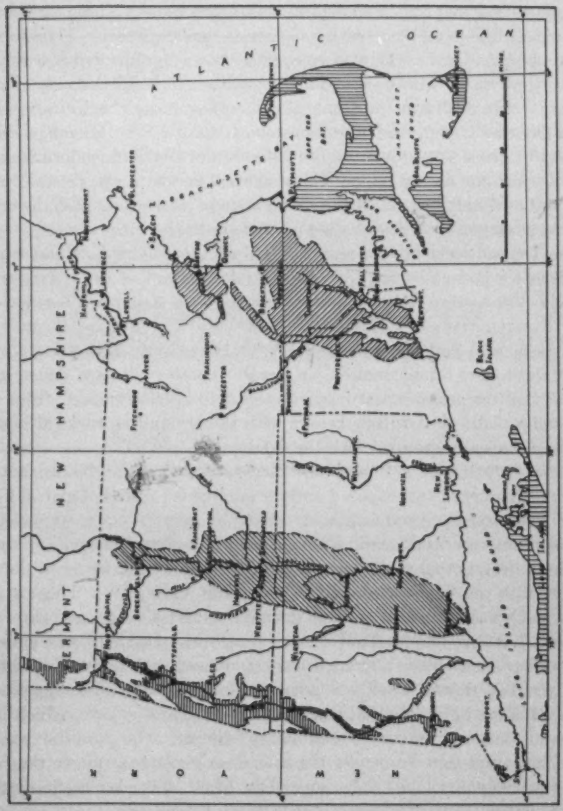
From a geological point of view, we may divide the study of groundwater into two parts: Groundwater in the hard rocks and groundwater in the unconsolidated deposits. In New England we have a great variety of hard rocks which contain more or less ground-water; incompletely blanketing these rocks in greater or less thickness is the glacial drift, which itself is extremely variable, and parts of which are excellent ground water reservoirs.

Groundwater in Hard Rocks.

We will first discuss groundwater in the hard rocks, although in many parts of New England this source of groundwater is not as important as the glacial drift. Water occurs in pores and cracks in hard rocks. Some rocks contain practically no pore space, and the groundwater is in cracks; other rocks contain much water in the pores. The percentage of pore space varies from practically nothing in some granites to 28 per cent. or more in some sandstones.[30]

In New England there is a great variety of rocks, which for our purposes can be divided into three groups. The first class of rocks comprises the porous rocks consisting of sandstones and conglomerates. A coarse sandstone or conglomerate in which the pores are not filled with natural cement has a large water capacity. Many of the New England sandstones and conglomerates, however, are so dense that they contain little water in their pores. The limestones constitute the second class of rocks. These rocks are usually dense, with negligible porosity, but they are soluble. Water entering cracks in the stone gradually enlarges them, with the result that extensive caverns are often formed in limestone. In these caverns we may have true underground rivers, practically the only condition in which underground streams exist, as it is hardly correct to speak of the seepage through the pores of sandstone or through the fine cracks of granite as an underground river. The third group of rocks is made up of the crystalline rocks, including the gneisses, schists, slates, granites, diorites, felsite, trap, and similar rocks. These rocks are all very dense, and the pore space is negligible from the point of view of groundwater. They are, however, intersected by innumerable cracks, and water moves through these cracks near

*Consulting Geologist, Boston, Mass.



GENERALIZED GEOLOGICAL MAP OF SOUTHERN NEW ENGLAND.

FIG. 1.

the surface of the earth. At greater depth the cracks are closed by the high pressure encountered, and water cannot pass through them.

Location of the Different Hard Rocks. The accompanying map (Fig. 1) shows the geology of southern New England very much generalized.[26, 34] There are actually about a hundred varieties of the hard rocks in this region, most of which can be classed as crystalline rocks. These all behave in practically the same way with relation to groundwater. Therefore I have grouped the hard rocks in three classes, shown by four symbols on the map: (1) the Triassic sandstones and shales with interbedded sheets of trap rock, which extend down the Connecticut Valley from the Vermont line to Middletown, Conn., and then southwesterly to New Haven, Conn.; grouped with these sandstones are the sedimentary rocks in eastern Massachusetts, which are shown by a different symbol on the map; (2) the limestones in the western part of Massachusetts and Connecticut; (3) the crystalline rocks shown by the unshaded part of the map.

The Triassic rocks are the only rocks in southern New England which are sufficiently porous to carry an appreciable amount of groundwater in their pores. In eastern Massachusetts there are two structural basins containing sedimentary rocks of Carboniferous Age: the Boston Basin containing slate and conglomerate, and the Narragansett Basin containing shales, sandstones, conglomerates, and coal. These rocks are dense, and practically all the groundwater in them occurs in cracks; in fact, from the groundwater point of view they belong with the crystalline rocks although they are geologically grouped with the sandstones.

There is a fifth area — Cape Cod, Marthas Vineyard and Nantucket — in which no hard rocks are exposed and the surface is formed of glacial drift or other unconsolidated sediments.

Groundwater in the Triassic Rocks of the Connecticut Valley. We will now take up the actual conditions in these different rock areas in detail, beginning with the sandstones of the Connecticut Valley.[20, 23] These sandstones were deposited in a large basin during the Triassic Age, and they are conveniently referred to as the Triassic sandstones. In this basin gravels were first deposited. Then a great thickness of sand with occasional layers of clay was laid down. Finally a great thickness of clay was deposited. The gravels have been solidified into conglomerate and arkose, which is a coarse sandstone or conglomerate containing feldspar. The sands have been changed into sandstone known as Longmeadow sandstone due to the fact that it is typically developed at Longmeadow, Mass. The clay has hardened into shale known as the Chicopee shale.

While these sediments were being deposited, extensive mud flats existed on which dinosaurs and other prehistoric monsters roamed and left their tracks. As the mud hardened into shale, the footprints became preserved as fossils, and many of them have been found.

The long period of formation of these sediments was not, however, entirely one of quiet deposition, for at several different times there were

volcanic eruptions, and immense sheets of lava were poured out, which solidified into trap rock. After the formation of the conglomerate, sandstone, shale and trap, all these rocks were tilted so that they dip to the east with an average dip of 15 degrees from the horizontal. Since the formation of these rocks, they have suffered erosion for a very long time. The sheets of trap, being harder and more resistant than the sandstone, stand up as ridges, while the sandstones and shales form the lowlands of the Connecticut Valley. These rocks have been broken into great blocks by numerous faults, and the blocks have been tilted and moved, with the result that the trap ridges are not continuous but are broken and offset. Another result of the faulting is that the sandstone beds are not continuous for long distances, but are broken by faults.

These Triassic sandstones are relatively dense, and compared with other sandstones in other regions their porosity is small, but they do contain some groundwater in their pores. A large part of the groundwater content,

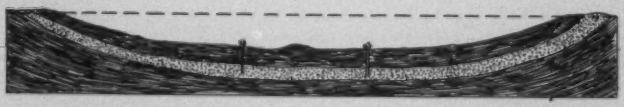


FIG. 2.

however, is in the numerous cracks. Since the sandstones occupy an elongated basin with hard crystalline rocks to the east and west, it might seem at first that ideal conditions existed for an artesian basin, but such is not the case.[27]

Ideal conditions for an artesian basin are shown in Figure 2. Such conditions occur in many places. There is shown in Figure 2 a layer of porous material, which might be either a layer of sand or of porous sandstone, underlain and overlain by impervious beds such as clay or shale. If this porous layer outcrops on high land and passes down under a valley, artesian flows will be produced when wells are driven in the valley through the shale into the porous sandstone. Water enters the porous beds in the highlands and seeps down through the porous bed under the lowlands, where it is under pressure and is prevented from escaping upwards by the overlying impervious beds. If wells are drilled in the lowlands through the impervious shale into sandstone, the water in the sandstone, being under a high head, will seek the path of least resistance, rise up in the wells, and flow out on the surface with greater or less force depending upon the difference in elevation between the intake and the outlet, and the freedom with which water passes through the porous bed.[29]

Artesian conditions often occur in slightly different ways. For instance, sloping eastward from the Rocky Mountains under the Great Plains are a series of sandstones and shales. Some of these sandstones are very porous and carry large amounts of water out under the plains, the water being prevented from rising to the surface by the overlying impervious beds of shale. When wells were first drilled through the shale into the sandstone, the water would rise to the surface, but in some areas so many wells have been drilled and so much water taken out of the sandstone that the wells no longer flow but have to be pumped.

Similar conditions exist on the New Jersey coastal plain. Alternating layers of sand and clay underlie this plain and slope from its inner border, which is practically along a line from Philadelphia to New York City, out towards the Atlantic Ocean. Water enters the outcrops of the porous sands and seeps down under the eastern part of New Jersey. Wells drilled along the coast, even out on the barrier beaches, as at Atlantic City where there is

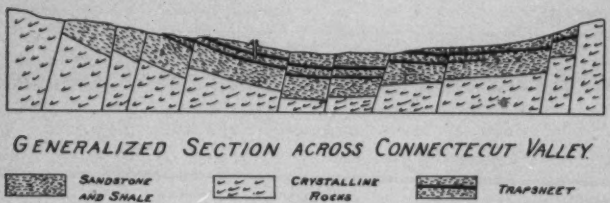


FIG. 3.

salt water on both sides, penetrate the overlying clays and enter the water-bearing sands. Originally these wells flowed, but so much water has been taken from the sand layer that the pressure in it has been reduced, and now the wells must be pumped.[57]

Figure 3 shows the conditions that actually exist in the Triassic basin of the Connecticut Valley. It is easily seen why we do not have artesian conditions. It is true we have layers of sandstone and beds of shale, but these shale beds are much broken up and are discontinuous, and water readily escapes through the cracks. Also the individual beds of sandstone are broken and discontinuous, and it is therefore impossible for water to travel from the highlands on either side into the center of the valley, because water soon comes to the end of any pervious bed and escapes to the surface through the numerous fault cracks. In some of the individual blocks which are surrounded by faults it is true that artesian conditions may exist on a small scale. (See Fig. 3). In Berlin, Conn., a well 117 ft. deep flowed at a rate of 100 g.p.m., and a flowing well in South Manchester yielded 250 g.p.m.

The Triassic sandstones of the Connecticut Valley are, however, an

important source of groundwater. Some of it occurs in the pores of the rock, but much of it occurs in the innumerable cracks which intersect these rocks.[47] Groundwater is found in all the rocks of the Connecticut Valley, even the dense trap. It is most abundant in the sandstone, less so in the shales, and still less so in the conglomerates and arkose which underlie the sandstones. Many successful wells have been drilled in these Triassic rocks, but there have been cases where a well close to a successful one has been a failure. The probability of at least a small yield for wells in the Triassic sandstone area of Connecticut has been estimated at about 94 per cent.

In general it appears that if water is not found within 250 or 300 ft. of the surface of the rock, it is unlikely that much can be obtained by going to greater depths. A well at Northampton was sunk to a depth of 3 700 ft. in the dense conglomerate already referred to, without striking water. In New Haven, a well 4 000 ft. deep was dry except for surface leakage. On the other hand many relatively shallow wells have been very successful. A well, 113 ft. deep, near Windsor, Conn., yielded 50 g.p.m., and in Hartford several wells about 200 ft. deep produced from 120 to 150 g.p.m. each. In Meriden, Conn., a well 1 000 ft. deep yielded 50 g.p.m., but a well only 300 ft. deep gave 170 g.p.m., or over three times as much. This water came from cracks, and the shallower well evidently intercepted more cracks in the rock than did the deeper well.

The conglomerate and arkose contain the least groundwater of any of the Triassic rocks. The deep wells which have been failures were in these rocks, and, in general, wells in the conglomerate and arkose are unsatisfactory. The scarcity of water in the conglomerate and arkose is due to their density and consequent lack of pore space and to the fact that they do not contain as many cracks as the trap and some other dense rocks.

The yield of wells in the sandstone varies greatly. Some have been dry, many have small yields of only a few gallons a minute, and some have large yields. A well in East Hartford yielded 265 g.p.m.; one in Hartford had a yield of 400 g.p.m.; and a well in South Manchester, Conn., flowed at a rate of 250 g.p.m. and has yielded up to 650 g.p.m. by pumping. Many other wells in the sandstone have been successful in varying degrees.

Trap is a crystalline rock, but for convenience the Triassic traps will be discussed at this time. We should expect to find very little water in the dense trap rocks, but on account of the way in which they are minutely cracked, some successful wells have been drilled in the trap. In Berlin, Conn., a well in trap rock flowed at the rate of 100 g.p.m.

Shales are theoretically entirely unsuited for yielding groundwater, but in the Chicopee shale there was once considerable disseminated lime which has since dissolved out, leaving small cavities. This is believed to explain the fact that there are some very successful wells in this shale. At Holyoke two wells, each 720 ft. deep, each yielded 450 g.p.m. by pumping.

The conditions probably existing in these Triassic rocks can be forecast by a geological study. For example, it is known that the chance of successful

wells in the conglomerate is very poor, and the position of this conglomerate can often be predicted even when it is not exposed at the surface. Also, since much of the water in the Triassic rocks is in joint cracks or faults, the location and direction of these can be determined. Thus a well can be located with the best chance of success.

The water from these Triassic rocks is generally hard,— often too hard to be satisfactorily used in boilers. Groundwaters from the trap are comparatively low in mineral matter, but those from the shales have a relatively high mineral content. Although there are many successful wells in these Triassic rocks, none of them yield very large amounts of water, such as several thousand gallons a minute, an amount which is often obtained from wells in the loose porous beds of the coastal plain. Although the groundwater from these Triassic rocks is generally hard, it is used extensively and is a valuable source of supply for the Connecticut Valley lowland. A small area of Triassic rocks occurs in the towns of Bethlehem, Woodbury, and Southbury in the highlands west of the Connecticut Valley. The groundwater conditions in this area have been very carefully studied.[40]

Groundwater in the Boston and Narragansett Basins. Turning again to Figure 1, the generalized geological map of southern New England, we will briefly consider the basins of sedimentary rocks in eastern Massachusetts and Rhode Island: the Boston Basin and the Narragansett Basin.

In the Boston Basin are slate and conglomerate, which were deposited in water in the Carboniferous Period, and also volcanic rocks.[38] These rocks are generally so dense that from the groundwater viewpoint they belong with the crystalline rocks. The slate in the Boston Basin known as the Cambridge slate is more massive, with less highly developed cleavage, than many slates. Wells in the Cambridge slate are often not satisfactory, but some have been successful, and yields of over 50 g.p.m. have been reported. On the other hand, in the conglomerate — which is known as the Roxbury conglomerate and also as Roxbury puddingstone — there have been many successful wells, and yields up to 100 g.p.m. have been obtained. In the Roxbury conglomerate, which is a very dense rock, practically all the water occurs in the cracks. From the groundwater viewpoint the rock should be classed with the crystalline rocks.

The Narragansett Basin contains shale, sandstone, conglomerate, and coal, all of Carboniferous age. Wells in the shale have yielded up to 100 g.p.m., and there are successful wells in the conglomerate. The sedimentary rocks of this basin, as well as those in the Boston Basin, are generally so dense that practically all the water occurs in cracks. From the groundwater viewpoint, therefore, they belong with the crystalline rocks.

Groundwater in the Limestone Areas of New England. Turning now to western New England, we will consider the conditions in the limestone areas. As can be seen on the map in Figure 1, there are irregular and broken belts of limestone extending northward from southern Connecticut up through Massachusetts. They also continue the length of Vermont,

where they are more important than in Massachusetts.[19] This limestone was deposited in the sea during the Cambrian and Ordovician Ages, and the rocks have since been folded, faulted, and uplifted. Erosion has carved the valley of western Massachusetts and Vermont in this limestone. Parts of the limestone have been metamorphosed and crystallized into marble, which might be considered with the crystalline rocks, but since the marble as well as the limestone is soluble, they are similar from the groundwater viewpoint. There are also areas of limestone in Maine and small areas of it elsewhere in New England. Since limestone is soluble, the cracks in the rock often become enlarged into caves and extensive caverns.

Caves are found in the limestone in western New England, but extensive caverns such as occur in the limestones of the South and elsewhere are unknown here. It is evident, however, that many of the cracks in the limestone have been enlarged by solution until they are effective passages for the movement of underground water. Wells in the limestone areas often intersect these passages and capture supplies of water. The conditions, however, are erratic, and there is no assurance that a given well will intersect a water passage. Since these enlarged cracks are very numerous, a well several hundred feet deep will usually obtain water in the limestone. A well, 400 ft. deep, in North Adams yielded 625 g.p.m., but the water was very hard. A drilled well in Canaan Village, Conn., produced only 17 g.p.m., and three other wells in Canaan yielded from 1.5 to 3 g.p.m. A well 386 ft. deep in Thomaston, Maine, yielded only 3 to 5 g.p.m. Some wells in the limestone belt of western Vermont yielded over 100 g.p.m. while others have been failures. Groundwater from limestone is, of course, generally hard, and it is more subject to pollution than groundwater from other rocks.

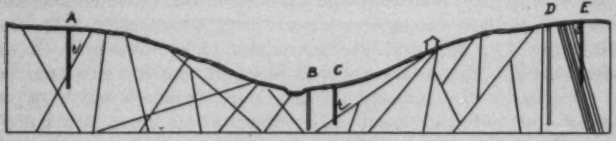
Water traveling through the pores of sandstone is effectively filtered, but such is not the case with water traveling through the open passages in limestone. I have often seen in the limestone areas of the South places where the outhouses of a farm were conveniently located over a sink hole or opening into a cave. This is an effective way of disposing of the sewage as far as the farmer is concerned, but the wastes often go directly into an underground stream in which they flow without being filtered, and probably come out in a large spring at some nearby point. In fact, it is sometimes possible to trace the line of underground drainage from a given sinkhole to the spring outlet. In such cases it is obvious that the spring, although it may appear pure, will actually be dangerously polluted. These springs are, however, often used for drinking water. Since the underground drainage in the limestone areas is less developed in Massachusetts than in the South, this danger may not be so extreme, but it should be remembered that water traveling through limestone is not filtered and may be polluted.

Groundwater in the Crystalline Rocks. We come now to the third class of rocks, the crystalline rocks, which outcrop in all of New England except in the relatively small areas of sedimentary rocks and limestone, and the areas where hard rocks are completely buried. The crystalline rocks include

the granites, diorites, gabbros, traps, felsites, and other igneous rocks, and also metamorphic rocks such as quartzites, gneisses, schists, and slates. Marbles might be included, but since they are soluble, they have been considered with the limestones.

In all the crystalline rocks, groundwater exists only in cracks. There are, however, many successful wells in the crystalline rocks, and the great majority of wells in these rocks give at least a small supply, but there is always the chance that a well will not intersect any water-bearing cracks and will be a failure.[22]

We may take the granites as typical of the crystalline rocks, and Figure 4 illustrates conditions which may exist in the granites. We have at A on a hill a well which intersects several joint cracks and from which water can be obtained by pumping, but the well will not be a flowing well since the intake of the cracks is at practically the same elevation as the top of the well. At B, a well was drilled in the valley but happened to be in a massive part



WELLS IN JOINTED GRANITE

FIG. 4.

of the rock, did not intersect any cracks, and was therefore a dry well. Well C, however, intersected a crack whose head was higher than the outlet of the well, and it was therefore a flowing well, but the water is subject to pollution from the house near the intake of this crack. On the hill to the right, Well D was in a great block of granite which was not intersected by any water-bearing cracks, and the well was a failure; but just beyond it we have Well E intersecting a fracture zone from which a plentiful supply of water can be pumped, but the well will not flow. This diagram shows the various possible conditions but is not intended to give any idea of the relative proportion of successful wells and failures. In granite 80 or 90 per cent. of the wells are usually successful, at least on a small scale.[12] As is shown in Figure 4, artesian or flowing wells may exist in crystalline rocks if the well intersects a system of joint cracks or a fault the intake of which is higher than the top of the well. Several artesian wells at Dalton, Mass., are believed to obtain their water from a large fault.[25]

Since the water in granite and all the other crystalline rocks occurs in cracks, the location of these cracks, and especially of faults and fracture zones, is important and can best be done by a geological study. Zones of

close joint cracks or fracture zones can be detected, and wells in these close-jointed zones are much more likely to be successful than wells in massive parts of the rock. Usually joint cracks in rocks have a more or less systematic arrangement, and it may be possible to determine the source of the groundwater which is obtained by a well and to avoid polluted supplies. For example, if there is a source of pollution near a fault crack and a well intersects this crack, the well will yield polluted water. Such a case is illustrated in Figure 4. The pollution of a well in that location could have been forecast. In some cases the source of pollution may be at a considerable distance, even in another watershed, but a study of the rock structure should indicate the danger of pollution. Thus a geological study is useful not only in increasing the chance of success by the proper location of a well but also in reducing the danger of obtaining polluted water.

Large flows of water are seldom obtained from wells in granite, but where a small supply is satisfactory, there is a high probability of success, and the water is usually excellent in quality. The successful wells in granite usually encounter water at less than 200 ft. from the surface, and it is generally found that it does not pay to drill much deeper than 200 ft. Of 80 wells in granite, 8 were failures, 29 were successful with the exact yield unknown, 16 gave 1 to 5 g.p.m., 6 yielded 6 to 10 g.p.m., 18 produced 11 to 50 g.p.m., and 3 were good for over 50 g.p.m. A well in Bridgton, Me., yielded 90 g.p.m.; one at Southwest Harbor gave 100 g.p.m.

Analyses of seven samples of water from wells in granite showed that the total solids varied from 81 to 419, and averaged 219 p.p.m. The granite waters are generally excellent for drinking purposes and satisfactory for other uses.

As indicated by the records of 117 wells in crystalline rocks, the average yield of wells in granite, gneiss, and schists (other than quartzite schist) is about the same. The yield in quartzite schist is but little more than half as much, and the yield in granodiorite is two and a half times as great.[22] Of these 117 wells only 13 were in granodiorite. Hence a larger number of records is needed to prove that granodiorite is usually a better source of groundwater than the other granitic rocks. A well in granodiorite at Stamford, Conn., yielded 200 g.p.m., but other wells in the same rock and in the same town had small yields. The amount of water in crystalline rocks varies with the thickness of cover of glacial drift. Where bare rock is exposed, rainwater runs off rapidly, but where the mantle of drift is thick, much of the rainwater is absorbed and transmitted into the cracks of the rocks.

The slates of the Boston Basin have been mentioned. In these massive slates successful wells are not numerous, but such is not the case with all slates. There are large areas underlain by slate in Maine in which successful wells are the rule, and moderate supplies of excellent water can usually be obtained. Water appears to occur at greater depths in these rocks than in the granite, and it has often been found worth while to go down to depths of 500 ft. Of 208 wells in slate 14 were failures, 4 gave water unfit for use,

90 were successful with the exact yield not known, 43 gave from 1 to 5 g.p.m., 14 yielded 6 to 10 g.p.m., 37 produced 11 to 50 g.p.m., 5 yielded 50 to 100 g.p.m., and 1 gave over 100 g.p.m. The yield of a 225-ft. well in slate at Cape Jellison was 200 g.p.m. These slates generally stand on edge and are intersected by innumerable cleavage cracks as well as by many joint cracks. Water enters these cracks readily and occupies them. On account of the excessive number of these cracks these slates may have a larger capacity for water than the granites or other crystalline rocks. The slates in northern Maine are generally calcareous, and the waters are hard; but the slates in southern Maine are much less calcareous, and the waters are not so likely to be hard. Groundwaters from these slates generally contain a lower amount of total solids than the granite waters, but the slate waters are harder and contain more lime and magnesium. When not too hard these slate waters are excellent.[10, 11]

Occasionally artesian conditions occur on a small scale in slate. If the intake of a system of joint cracks is at a higher elevation than the top of the well which intersects the cracks, artesian flow will result. Such a condition was encountered in a well at Winslow, Me.[39]

Groundwater in the Unconsolidated Deposits.

Having covered briefly the most common groundwater conditions in the hard rocks, we will now turn to consideration of the groundwater in the unconsolidated deposits. As we have seen before on the map (Fig. 1) there is a large area on Cape Cod, Marthas Vineyard and Nantucket where no hard rocks are exposed at the surface, and where the depth to hard rock is great. In this area conditions are somewhat different from the rest of New England. Although the surface of Cape Cod and the adjacent islands is largely composed of glacial drift, it is underlain by unconsolidated sedimentary deposits, gravels, sands and clays of Cretaceous and Tertiary ages similar to the coastal plain deposits of New Jersey and the southern states. These deposits are known to occur on Marthas Vineyard and on the mainland north of the Cape Cod Canal. They probably underlie also Nantucket and Cape Cod. Relatively little is known about the continuity of these sedimentary formations of Cape Cod and the islands. Since the blanket of glacial drift is generally thick and usually contains a large amount of groundwater near the surface, there has been little occasion to drill through the drift into the underlying coastal plain formations. It is possible, however, that if the increase of population makes it necessary, considerable supplies of water may be found in these formations below the glacial drift.

In New England, outside of Cape Cod and the adjacent islands, glacial drift forms the surface except where hard rocks are exposed. This blanket of drift varies in thickness, but it averages thin and nowhere approaches the thickness of the unconsolidated sediments on Cape Cod and the islands.

The Glacial Drift. Groundwater in the glacial drift is very important

in New England. It occurs in varying quantities in the different types of glacial deposits and can be obtained in large quantities from some of them. Therefore an understanding of the types of glacial deposits is desirable.

The great ice sheet which once covered all of New England moved slowly but irresistibly from north to south, scraping up the soil and loose fragments, tearing off blocks of rock, and grinding solid rock to rock flour. Thus an enormous amount of debris became incorporated in the ice sheet. When the ice finally melted, this debris was deposited in different ways over the surface of the country.

The term glacial drift includes all the varieties of debris which were deposited from or washed from the great ice sheet. There are, however, two distinctly different classes of glacial drift. The first is the unstratified, unwashed till which is a heterogeneous mixture of everything from clay to boulders. In this region where the till commonly contains considerable amounts of clay and numerous boulders, it is called boulder clay. This material is also commonly referred to as hardpan. The boulder clay is dense and relatively impervious. It does, however, contain groundwater, and groundwater can move through it very slowly. Small supplies of water, such as for a single farm, can usually be taken from wells in the boulder clay, but large supplies of water cannot be obtained from this material. The boulder clay forms an incomplete blanket on the bed rock and is usually thin in the valleys and thicker on the hills. In the valleys the boulder clay is frequently overlain with other materials.

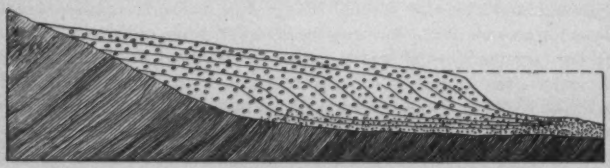
The other great class of glacial drift is stratified drift which has been washed and sorted by water. This includes deposits of gravel, sand, and clay. The clay deposits are, of course, practically impervious, and satisfactory groundwater supplies cannot be obtained from them. Large quantities of groundwater occur, however, in the deposits of sand and gravel, and there are several kinds of these deposits. Some of these deposits which are important from the groundwater viewpoint are valley or flood plain deposits of glacial and recent streams, outwash plains, deltas, and eskers.

Most valleys in the glaciated regions are floored with varying thicknesses of stratified drift. Sometimes this was deposited in deltas in lakes, which will be described later. Sometimes stratified sand and gravel were deposited in the stream beds, building them up and forming deposits of pervious material in the bottoms of the valleys. Important groundwater supplies occur in these deposits, and many communities obtain their supplies from wells in the valley fill. The new water supply for Concord, N. H., is from wells in the glacial deposits in the Merrimack Valley. The glacial deposits of the Connecticut Valley form an important underground storage reservoir, in which there are many successful wells.

Where the front of the ice sheet stood for a long time, outwash plains of sand and gravel were built up from the detritus washed from the ice sheet. The distinction between outwash plains and deltas is that the first were deposited on the land and the latter were deposited in water. There

are consequently differences in structure and form. When the ice advanced and then retreated, ridges of debris, known as moraines, were formed, and outwash plains were deposited in front of these. The farthest advance of the ice is marked by a moraine which extends through the center of Long Island out to Montauk Point, then under the sea to Block Island, Marthas Vineyard, and Nantucket. Another advance of the ice is marked by a moraine, parts of which can be seen along the north shore of Long Island, the south shore of Rhode Island, and on Cape Cod. Outwash plains extend southward from both of these moraines. These outwash plains are important groundwater reservoirs, and there are many wells in them. Much of the water supply of Cape Cod, Marthas Vineyard, and Nantucket comes from wells in the outwash plains. At many other places in New England smaller outwash plains occur and are the source of important groundwater supplies.

When a glacial stream discharged into a lake, a delta was built. A delta consists of three parts, as can be seen in Figure 5. Currents carry the grains



SECTION OF A TYPICAL GLACIAL DELTA.

FIG. 5.

of sand out across the delta and drop them down on the inclined front surface, forming inclined beds known as foreset beds. Finer material is swept out in front of the delta and is deposited on the bottom of the lake, forming horizontal layers known as bottomset beds. As the delta is built up to the surface of the water, coarse material is deposited on the top in nearly horizontal layers known as topset beds. The diagram shows the water surface of the lake at the time the delta was built, but the lake has long since been drained, and the delta now forms a terrace in a valley. The topset beds form an open surface which rapidly absorbs the rainfall; the foreset beds form a large body of sand capable of storing a large amount of groundwater; the underlying bottomset beds form an impervious floor which prevents easy escape of the water, with the result that we often have in these deltas excellent conditions for groundwater supplies.[17] The delta shown in the diagram is small, but glacial deltas are often a mile, or several miles, across. Deltas usually have definite characteristics which are easily detected by a geological investigation.

Streams flowing on or in the ice deposited sand and gravel in their beds, and when the ice melted away these deposits of sand and gravel were

dropped on the surface of the ground, forming long winding ridges known as eskers. These are composed of sand and gravel, often very coarse. Where they rise high above the general surface, they do not contain groundwater, as it easily drains from them. There are, however, many cases where eskers in the valleys have been buried by later deposits of clay or silt, and it is sometimes possible to obtain a satisfactory groundwater supply by drilling through the clay or silt into one of these eskers. The buried eskers may be concealed, and it is sometimes a difficult geological problem to locate them.

All the glacial deposits except the boulder clay have a definite structure, and this structure controls the movement of groundwater. If this structure is understood, the groundwater conditions can often be forecast, and in many cases the source of the groundwater can be determined. Thus the chance of dry wells or of polluted water can be reduced.

Buried Valleys. As the ice melted away from New England, large quantities of drift were swept down the valleys, filling them to greater or less depths. Where this filling consists of sand and gravel, satisfactory groundwater supplies can be obtained. Prior to the glaciation of New England the drainage system of New England was very different from that which exists today. As the ice melted away, great quantities of glacial drift were deposited irregularly over the surface of the country, blocking many of the valleys. When the streams again attempted to flow to the sea, they often found their old courses blocked and were forced to take new courses across the surface of the drift. As a result of these conditions there are numerous buried valleys in New England. Many of these are known, and undoubtedly there are others which are not. We may mention one typical example — the preglacial course of the Merrimack. There is good evidence that in preglacial times the Merrimack did not flow to the sea via Lawrence, Haverhill, and Newburyport, as now, but that it continued southeasterly by the site of Boston into Boston Bay. The Mystic Lakes, Spy Pond and Fresh Pond lie in this buried valley.[13] In some cases these preglacial valleys were filled with till or clay, in which case satisfactory groundwater supplies cannot be obtained. Generally, however, the filling consists of sand and gravel, and these valleys therefore form great rock basins filled with sand and gravel which are saturated with water. The wells which supply Lowell are in the preglacial valley of the Merrimack, as are wells in Winchester and Woburn. A deep bed rock valley near Roxbury Crossing in Boston is filled with sand and gravel, and several successful wells were drilled in it. This good groundwater supply is probably one reason why so many breweries were located in that vicinity. These buried valleys often offer excellent groundwater supplies. The location of these valleys, therefore, becomes of importance. Fortunately, it is often possible to find these buried valleys by a geological study.

Absence of Underground Rivers. From this brief survey of groundwater conditions it can be seen that underground rivers flowing across New England, of which we occasionally hear rumors, are an impossibility. The

idea that underground streams flow from the White Mountains to southern New England is entirely unfounded. An understanding of the geological conditions shows that such underground rivers cannot exist. Such streams would be obliged to flow most of the way through the dense crystalline rocks. It has already been shown that true underground rivers exist only in the limestones. Even where such underground streams are highly developed, they usually do not travel very long distances underground. In New England where the limestone areas are less extensive and the underground drainage less highly developed, extensive underground rivers even in the limestone are highly improbable.

Under the Coastal Plain and under the Great Plains, water does travel long distances through porous beds, but such slow seepage cannot be called an underground river. Even such underground flows do not exist for long distances in New England, for here the deposits of sand and gravel are much less extensive than under the Coastal Plain. The most extensive and continuous deposits of unconsolidated materials occur on Cape Cod, the adjoining mainland, and the adjacent islands, that is, the area shown on the map in Figure 1. As has been explained, most of the wells in this area are shallow, and little is known about the movements of groundwater at depth.

Groundwater moves slowly through the sands and gravels in the buried river valleys. It may travel considerable distances in valleys such as those of the Connecticut and Merrimack rivers, but often parts of the valleys are filled with clay or boulder clay which restrict the movement of underground water.

In brief, underground rivers are non-existent in New England except possibly in the limestone areas, and extensive concentrated underground flows through the sands are scarce. It is certain that streams of water do not flow underground from one part of New England to another, and the idea that enormous supplies of water can be obtained by tapping an underground river is erroneous. As has been shown, important supplies of groundwater can be developed in New England, but they do not come from underground rivers.

Conclusion.

Some of the varied groundwater conditions of New England have been discussed. More has been said about groundwater in the hard rocks than in the unconsolidated deposits because the conditions in the latter vary so greatly from place to place that a general description of conditions is difficult. Each locality is a special problem requiring investigation in the field. The more important types of unconsolidated deposits and the groundwater conditions in them have, however, been described.

The glacial deposits are probably more important than the hard rocks as sources of groundwater in New England, and larger yields can be obtained from them. On the other hand, satisfactory supplies of water can usually be obtained from the hard rocks where glacial deposits are lacking

or where they consist of materials from which satisfactory supplies cannot be obtained.

The groundwater conditions of New England may be summed up as follows: Wells in the hard rocks generally yield some water and in some cases large supplies have been obtained. Small supplies of groundwater, sufficient for a single family, can usually be obtained from a well in boulder clay, but large supplies cannot be derived from this material. Wells in clay are seldom satisfactory. Large supplies of groundwater can often be obtained from the deposits of sand and gravel, but this is not always so.

Both in the case of the hard rocks and of the glacial deposits much can be gained by an adequate geological investigation before the wells are placed. In this way the best location can be determined and the chance of failure and the danger of obtaining polluted water can be reduced.

Some of the best types of groundwater supplies, such as those which occur on the Coastal Plain, are not well represented in New England, but we have here many excellent groundwater supplies.

Selected References.

A list of the more important articles on the groundwater of New England and on the geology of those areas discussed in this paper are given below. Most of the articles on groundwater are of a very general nature. Only a few areas have been described with any detail. More detailed work has been done in Connecticut than in the other New England states. Although the geology of New England has been studied for over a century, there are large areas which have never been adequately described, and some of the older articles are of little use now. The geological articles listed here include general articles, with geologic maps, covering the states of Connecticut, Massachusetts, and Rhode Island, and articles covering the areas discussed in this paper.

[1] Bastin, Edson S., Description of the Rockland, Maine, Quadrangle, U.S.G.S. Folio **158**, 15 pp., maps, (1908). Gives a description of the geology and the groundwater conditions.

[2] Bastin, E. S., and Williams, H. S., Description of the Eastport, Maine, Quadrangle, U.S.G.S. Folio **192**, (1914). Gives a description of the geology with a note on water supplies.

[3] Bayley, W. S., Notes on Groundwater Resources of Maine, U.S.G.S. Water-supply Paper **102**, 27–55, (1904).

[4] Bayley, W. S., Underground Waters of Maine, U.S.G.S. Water-supply Paper **102**, 40–56, (1905).

[5] Bayley, W. S., Records of Deep Wells in Southern Maine, U.S.G.S. Water-supply Paper **223**, 238–257, (1909).

[6] Boutwell, J. M., Notes on the Wells, Springs, and General Water Resources of New Hampshire, U.S.G.S. Water-supply Paper **102**, 56–72, (1904).

[7] Brown, J. S., A Study of Coastal Groundwater with Special Reference to Connecticut, U.S.G.S. Water-supply Paper **537**, 101 pp., map, (1925).

[8] Brown, J. S., Ground Water in the New Haven Area, Connecticut, U.S.G.S. Water-supply Paper **540**, 206 pp., maps, (1928).

[9] Campbell, E. W., Public Water Supplies of Maine, Jour. N.E.W.W.A., Vol. **41**, 99–128, (1927). Lists the type of supply of the various communities.

[10] Clapp, F. G., Underground Waters of Southern Maine, U.S.G.S. Water-supply Paper **223,** 268 pp., geologic map, (1909).

[11] Clapp, F. G., Occurrence and Composition of the Well Waters in the Slates of Maine, U.S.G.S. Water-supply Paper **258,** 32–39, (1911).

[12] Clapp, F. G., Occurrence and Composition of the Well Waters in the Granites of New England, U.S.G.S. Water-supply Paper **258,** 40–47, (1911).

[13] Crosby, W. O., A Study of the Geology of the Charles River Estuary and the Formation of Boston Harbor, *Report of the Committee on the Charles River Dam,* 345–369, (1903). A brief account of the surface geology. Describes the preglacial course of the Merrimack. See especially p. 351.

[14] Crosby, W. O., and La Forge, Laurence, Notes on the Wells, Springs, and General Water Resources of Massachusetts, U.S.G.S. Water-supply Paper **102,** 94–117, (1904).

[15] Crosby, W. O., Notes on the Wells, Springs, and General Water Resources of Rhode Island, U.S.G.S. Water-supply Paper **102,** 119–125, (1904).

[16] Crosby, W. O., Underground Water Conditions in Massachusetts and Rhode Island, U.S.G.S. Water-supply Paper **114,** 68–75, (1905).

[17] Crosby, W. O., Water Supply from the Delta Type of Sand Plain, U.S.G.S. Water-supply Paper **145,** 161–178, (1905).

[18] Dale, T. N., Water Resources of the Fort Ticonderoga Quadrangle, Vermont and New York, U.S.G.S. Water-supply Paper **110,** 126–129, (1905).

[19] Dale, T. N., The Lime Belt of Massachusetts and Parts of Eastern New York and Western Connecticut, U.S.G.S. Bulletin **744,** 71 pp., (1923). Describes the geology of the limestone area of western Massachusetts and Connecticut.

[20] Davis, W. M., The Triassic Formation of Connecticut, U.S.G.S. Eighteenth Annual Report, Part II, 1–192, map, (1898). A detailed description of the Connecticut Lowland in the State of Connecticut.

[21] Ellis, A. J., Groundwater in the Waterbury Area, Connecticut, U.S.G.S. Water-supply Paper **397,** 73 pp., (1916).

[22] Ellis, E. E., Occurrence of Water in Crystalline Rocks, U.S.G.S. Water-supply Paper **160,** 19–28, (1906). See especially p. 27.

[23] Emerson, B. K., Geology of Old Hampshire County, Massachusetts, U.S.G.S. Monograph **29,** 790 pp., (1898). A detailed description of the geology of the Connecticut Valley and adjacent areas.

[24] Emerson, B. K., Outlines of the Geology of Western Massachusetts, U.S.G.S. Folio **50,** 8 pp., maps, (1898). Describes in briefer form than Monograph 29 the geology of the Holyoke Quadrangle.

[25] Emerson, B. K., Geology of Eastern Berkshire County, U.S.G.S. Bulletin **159,** 139 pp., (1899). Describes the artesian wells at Dalton and gives detailed description of the geology. See especially pp. 90–92.

[26] Emerson, B. K., Geology of Massachusetts and Rhode Island, U.S.G.S. Bulletin **597,** 289 pp., map, (1917). See this for detailed geological map.

[27] Fuller, M. L., Triassic Rocks of the Connecticut Valley as a Source of Water Supply, U.S.G.S. Water-supply Paper **110,** 95–112, (1905).

[28] Fuller, M. L., Underground Waters of New Hampshire, U.S.G.S. Water-supply Paper **114,** 57–59, (1905).

[29] Fuller, M. L., Significance of the Term Artesian, U.S.G.S. Water-supply Paper **160,** 9–15, (1906).

[30] Fuller, M. L., Total Amount of Free Water in the Earth's Crust, U.S.G.S. Water-supply Paper **160,** 59–72, (1906). See especially p. 61.

[31] Gage, S. DeM., Description of Public Water Supplies of Rhode Island, Jour. N.E.W.W.A., **40,** 117–127, (1926). Lists the type of supply of the various communities.

[32] Goldthwait, J. W., The Geology of New Hampshire, New Hampshire Academy of Science Handbook **1,** 86 pp., maps, (1925). A brief description of the geology of the State.

[33] Gregory, H. E., The Geology of Connecticut in Relation to its Water Supply, Connecticut Board of Agriculture, Annual Report **39,** 283–297, (1906).

[34] Gregory, H. E., and Robinson, H. H., Preliminary Geological Map of Connecticut, Conn. Geol. Surv. Bulletin 7, 39 pp., map, (1907). Geological map with brief description of formations.

[35] Gregory, H. E., Underground Water Resources of Connecticut (With A Study of the Occurrence of Water in Crystalline Rocks, by E. E. Ellis), U.S.G.S. Water-supply Paper **232,** 200 pp., (1909).

[36] Gregory, H. E., and Ellis, A. J., Groundwater in the Hartford, Stamford, Salisbury, Willimantic and Saybrook Areas, Connecticut, U.S.G.S. Water-supply Paper **374**, 150 pp., maps, (1916).

[37] Katz, F. J., Preliminary Report on the Geology of the Portland and Casco Bay Quadrangles, Third Annual Report of the Maine State Water Storage Commission, 170–184, (1913).

[38] La Forge, Laurence, Geology of the Boston Area, Massachusetts, U.S.G.S. Bulletin **839**, 105 pp., maps, (1932). Gives a description of the geology with a note on groundwater.

[39] Little, Homer P., A Flowing Artesian Well at Winslow, Maine, *Science*, **49**, 24, 25, (1919).

[40] Meinzer, O. E., and Stearns, N. D., A Study of the Groundwater in the Pomperaug Basin, Connecticut, U.S.G.S. Water-supply Paper **597-B**, 76–146, map, (1929). Description of groundwater conditions in Bethlehem, Woodbury, and Southbury.

[41] Palmer, H. S., Groundwater in the Southington-Granby Area, Connecticut, U.S.G.S. Water-supply Paper **466**, 219 pp., (1921).

[42] Palmer, H. S., Groundwater in the Norwalk, Suffield, and Glastonbury Areas, Connecticut, U.S.G.S. Water-supply Paper **470**, 171 pp., (1920).

[43] Perkins, G. H., A Summary of the Investigations upon the Drinking Waters of Vermont, Fourth Report of the Vermont State Geologist, 213–223, (1904).

[44] Perkins, G. H., Notes of Water Resources of Vermont, U.S.G.S. Water-supply Paper **102**, 73–93, (1904). Gives analyses and lists of wells and springs.

[45] Perkins, G. H., Underground Waters of Vermont, U.S.G.S. Water-supply Paper **114**, 60–67, (1904).

[46] Perkins, G. H., The Drinking Waters of Vermont, Fifth Report of the Vermont State Geologist, 254–344, (1906). Describes the supplies of the various communities.

[47] Pynchon, W. H. C., Drilled Wells of the Triassic Area of the Connecticut Valley, U.S.G.S. Water-supply Paper **110**, 65–94, (1905).

[48] Scott, W. J., and Almquist, F. O. A., Public Water Supplies of Connecticut, Jour. N.E.W.W.A., **43**, 298–317, (1929). Lists type of supply of the various communities.

[49] Sterling, C. I., Jr., and Belknap, J. B., Iron in the Groundwater Supplies of Massachusetts, *Jour. Boston Soc. Civ. Eng.*, 1–26, (1932).

[50] Stone, Guy C., The Glacial Gravels of Maine and their Associated Deposits, U.S.G.S. Monograph **34**, 499 pp., maps, (1899).

[51] Shaler, N. S., Woodworth, J. B., and Foerste, A. F., Geology of the Narragansett Basin, U.S.G.S. Monograph **33**, 402 pp., maps, (1899). Detailed description of the geology of the Narragansett Basin in Massachusetts and Rhode Island.

[52] Smith, Geo. O., Water Resources of the Portsmouth-York Region, New Hampshire-Maine, U.S.G.S. Water-supply Paper **145**, 120–128, (1905).

[53] Smith, Geo. O., Water Supply from Glacial Gravels near August, Maine, U.S.G.S. Water-supply Paper **145**, 156–160, (1905).

[54] Smith, Geo. O., Bastin, E. S., and Brown, C. W., Description of the Penobscot Bay Quadrangle, U.S.G.S. Folio **149**, 14 pp., maps, (1907). Describes the geology and the groundwater conditions.

[55] Suttie, R. H., Report on the Water Resources of Connecticut, Conn. State Geol. Surv. Bulletin **44**, 168 pp. (1928).

[56] Taylor, F. B., Water Resources of the Taconic Quadrangle, New York, Massachusetts, Vermont, U.S.G.S. Water-supply Paper **110**, 130–133, (1905).

[57] Thompson, David G., Groundwater Supplies of the Atlantic City Region, Department of Conservation and Development, State of New Jersey, Bulletin 30, 138 pp., (1928). See especially pp. 68–76.

[58] Waring, G. A., Groundwater in the Meriden Area, Connecticut, U.S.G.S. Water-supply Paper **449**, 83 pp., (1920).

[59] Statistics of Massachusetts Water Supplies, Jour. N.E.W.W.A., **38**, 144–197, (1924). Lists the type of supply of the various communities.

[60] Public Water Supply Control and Water Analyses, 31st Report of the State Board of Health of the State of New Hampshire, 17–96, (1930). Lists the type of supply of the various communities.

DISCUSSION.

GORDON M. FAIR.* May I ask Mr. Crosby where the intake area for the lower strata of Cape Cod would be located?

MR. CROSBY. On the map the northwestern edge of the belt of unconsolidated deposits extends across from Buzzards Bay to Plymouth. That line marks the northwestern limit of the area in which no hard rocks are exposed. Northwest of this boundary we have the hard rocks with the incomplete covering of glacial drift. Therefore the outcrop of the sedimentary beds which underlie Cape Cod is southeast of this line. If a bed outcrops and is continuous, it is possible that groundwater might be carried down under Cape Cod, and barely possible that it might be carried out under Marthas Vineyard, but if so, it would pass under Marthas Vineyard at very great depth — far below any of the wells that have been drilled there. I think that is the only case in New England where you might have a partial exception to the statement I made about water not traveling underground great distances. Any movement such as that would be a very slow seeping, however, quite different from what is generally meant in the stories about underground rivers. There may be something similar to the conditions on the coastal plain of New Jersey, but very little is known about it.

E. SHERMAN CHASE.† How about the developed wells of Long Island?

MR. CROSBY. It was once supposed that the water on Long Island came from Connecticut. More than twenty years ago a very extensive study of the groundwater conditions of Long Island was made by W. O. Crosby. His conclusions were that the water-bearing beds under Long Island are not continuous from under Connecticut. We do not have those coastal plain deposits exposed at all in Connecticut. They have been eroded, washed away from the Connecticut side of Long Island Sound, and the hard rocks, with a little veneer of glacial drift, are exposed right down to the Sound. The water-bearing beds which pass under Long Island probably come to the surface under the Sound unless they are concealed by impervious layers of clay. The groundwater of Long Island is entirely the water that falls as rain on Long Island. It is not possible to draw large supplies there from New England. It is possible that some of the water that might be found in the hard rocks on the western end of Long Island comes from the mainland, but there is not sufficient evidence to be sure about that.

PAUL F. HOWARD.‡ I would like to ask what would be the nature of the formation of the water-bearing strata under the Cape in which he would expect to find water. I was told that in Provincetown one well was driven some years ago 472 ft. below sea level. They found the same sand as near the surface, and they also encountered salt water for that depth.

MR. CROSBY. On Cape Cod, or, at least over most of Cape Cod, are the glacial deposits, and they go down to varying depths,— we do not know how far in some cases. Below them there are stratified layers of sand, of

*Associate Professor of Sanitary Engineering, Harvard University, Cambridge, Mass.
†Of Metcalf and Eddy, Engineers, Boston, Mass.
‡Of Whitman and Howard, Civil Engineers, Boston, Mass.

clay, and of gravel, which might not be very different from the glacial deposits. Provincetown being entirely surrounded by salt water, it would be exceedingly unlikely that there would be any fresh water which would get into those underlying layers except what falls on the surface and percolated. Salt water being denser seeps in under the fresh water, with the result that there is a basin of fresh water with salt water beneath and on the sides. If a well goes below the bottom of the fresh water, it will encounter salt water. Such conditions are known on many islands, and this is apparently the case at Provincetown.

ARTHUR D. WESTON.* Mr. Crosby's paper on the relation of geology to the groundwater supplies of New England is a much needed addition to the library of papers of this Association.

As Mr. Crosby has brought out, the wells in crystalline rocks in Massachusetts generally do not yield water in large quantities. Consequently, municipalities have found it more economical to obtain their supplies from surface sources or from wells in glacial drift. However, there are numerous private sources of water supply taken from wells in rock which yield water of excellent quality and of sufficient quantity for private use. There are but three municipalities in Massachusetts which take this water supply from wells in rock. Avon takes water from two wells some 200 ft. deep; North Adams has two deep wells in limestone 550 and 625 ft. deep and uses them as auxiliary sources of water supply. Adams takes water from six wells 106 ft. deep in limestone and uses them as auxiliary sources for the Adams Fire District.

The results of the analyses of water quality available to the Department of Public Health show that the hardness of the public groundwater supplies in Massachusetts varies from 5 p.p.m. in some of the shallow tubular wells in the glacial deposits of Cape Cod and the islands of Marthas Vineyard and Nantucket to about 225 p.p.m. in the deep wells in limestone used at times as sources of water supply in the city of North Adams. In general the waters taken from the ground in the westerly part of the state are harder than those in the easterly part, but some of the supplies in the former, especially those taken from glacial drift, are as soft as many of the waters in the latter.

None of the public surface water supplies in Massachusetts is very hard. The supplies in the Berkshire Valley are the hardest, although one of the softest of the surface waters is that of the nearby town of Hinsdale in the Berkshire Hills. It has a hardness of but 4 p.p.m. Throughout the state as a whole the hardness of the surface waters varies from 4 to about 90 p.p.m.

In the easterly part of the state, the waters of the public supplies are for the most part very soft except those taken from areas where there are deposits of iron. An exceptionally hard water in the easterly part of the state is that taken from a well used by the town of Marblehead. This

*Chief Engineer, State Department of Health, Boston, Mass.

water has a hardness of about 90 p.p.m. I have never heard a satisfactory explanation of this condition.

That part of Mr. Crosby's paper which makes reference to the groundwaters of Marthas Vineyard and Nantucket in which he says that statements to the effect that the groundwater on these islands has its source in the vicinity of the White Mountains are unfounded is particularly interesting to the writer. His conclusions are well substantiated by the analyses of the water of the public water supplies on these two islands. The waters on Marthas Vineyard have a hardness of but 5 p.p.m. and on Nantucket less than 15 p.p.m. It is obvious that waters as soft as these could not come through the ground from points as far distant as New Hampshire or even from the adjacent mainland where the groundwaters have a hardness from 25 to 40 p.p.m.

Mr. Crosby points out the danger of pollution of water taken from cracks and crevices in rocks and that the water from the hard crystalline rocks in Massachusetts is chiefly from these cracks. The matter of pollution of water from wells in rock is frequently called to the attention of the Department of Public Health. The Department found upon examination of a well north of Boston some 100 ft. deep that the water was polluted by pharmaceutical wastes discharged into a swamp from a manufacturing plant about 1 500 ft. away from this well. The consulting engineer engaged by the well owner in his case against the manufacturing company was able to dye cloth from the water pumped from the well after dyes had been placed on the edge of the swamp where it was polluted with the pharmaceutical wastes.

A very interesting study of the waters of deep wells in Boston and its vicinity was made by the State Board of Health in 1894. Much of the geological data supplied in connection with this study was obtained from Professor W. O. Crosby. At that time some 21 wells were examined in Roxbury, 3 in Cambridge, 7 in Charlestown, 7 in the South End of Boston and one in the Back Bay. These wells varied in depth from 28 to 1 200 ft. and the results of the analyses show that the water of many of the wells was affected by salt and had a hardness of from 50 to over 1 600 p.p.m. An examination of the analyses of the samples collected in connection with these wells published in the Annual Report of the State Board of Health for 1894 shows that the waters were affected by pollution of the groundwater in their vicinity.

The writer in the year 1906 examined some 160 wells in Boston and vicinity and on the islands in Boston Harbor. These varied in depth from 30 to 1 200 ft. and yielded from 3 to over 300 g.p.m. The results of the analyses confirmed those of the investigation of 1894. Many of these well waters had a high chloride content and were very hard; most of them were affected by pollution.

Mr. Crosby brings out in his paper very clearly that wells in our hard crystalline rocks can be depended on only to yield water from cracks.

Under the circumstances, water from such sources, while clear and colorless, must be viewed with suspicion. Engineering and geological studies made in connection with such water supplies would assist in preventing the use of wells in areas where water must obviously be affected by sources of pollution in the general vicinity. Careful geological observations will recognize the direction and dip of these cracks.

PROCEEDINGS.

JANUARY MEETING.

HOTEL BRUNSWICK, BOSTON, MASS.

January 11, 1933.

President Richard H. Ellis in the Chair.

Secretary Gifford. At the Executive Committee meeting this morning two new members were elected: Clarence I. Sterling, Jr., Junior Sanitary Engineer, Mass. Dept. of Public Health, Newtonville, Mass.; Truman M. Curry, Jr., Associate Engineer with Nicholas S. Hill, Jr., Consulting Engineer, Watertown, Conn.

At the meeting there was appointed: A Committee to Investigate and Report on Uniform Practice in Collections of Water Rates. This committee is to serve with a similar committee of the Associated Boards of Health.

ARTHUR D. WESTON.* The matter of shutting off water has become so important that the Commissioner of Health is asking the Associated Boards of Health to appoint a committee to go into that matter. I suggested that the Water Works Association appoint a similar committee and that the two committees meet at least once and report back to their organizations.

A paper "The Geology of Ground Water Supplies of New England," was read by Irving Crosby, Consulting Geologist, Boston. Messrs. Gordon M. Fair, E. Sherman Chase, and Paul F. Howard took part in the discussion.

A paper "Replacement of Brook Crossing with Bridge Crossing on 42-inch Pipe Line," was read by Harold W. Griswold, Deputy Manager and Deputy Chief Engineer, Water Bureau, Metropolitan District Commission, Hartford, Conn. Messrs. David A. Heffernan and Hervey A. Hanscom took part in the discussion.

(*Adjourned.*)

*Chief Engineer, Massachusetts State Department of Health, Boston, Mass.

FEBRUARY MEETING.

HOTEL BRUNSWICK, BOSTON, MASS.

Wednesday, February 8, 1933.

President Richard H. Ellis in the Chair.

Massachusetts House Bill No. 649 pending before the legislature was discussed by Charles W. Sherman.*

Secretary Gifford. At the Executive Committee meeting this morning two members were elected as follows: Francis W. Hamilton, Member of Brookline Water Board, Brookline, Mass.; Samuel M. Ellsworth, Consulting Engineer, Boston, Mass.

THE PRESIDENT. The credit for today's program is due almost entirely to Mr. Esty, and I am going to ask him to take charge of the meeting at this time.

(*Mr. Roger W. Esty takes the Chair for the balance of the meeting.*)

A paper "Marking of Hydrants," was read by Sidney S. Anthony, Engineer and Superintendent, Augusta Water District, Augusta, Maine.

A paper "Hydrant Pressure and Flowage Tests at Portland, Maine," was read by Leavitt R. Smith, Portland Water District, Portland, Maine.

These two papers were discussed by R. D. Chase, Henry F. Hughes, Horace J. Cook, Charles W. Sherman, James W. Graham, Donald C. Calderwood, Harry U. Fuller, and Arthur C. King.

A talk on "Jointing Compounds and Temperatures," was given by George C. Brehm, Waltham, Mass.

A paper "Joint Leakage Tests on 16-inch Ball and Socket Pipe at Haverhill, Mass.," was read by Herbert C. Crowell, Superintendent of Water Works, Haverhill, Mass.

The subject of jointing compounds and leakage was discussed by Messrs. Patrick Gear, Albert A. Ross, Milton Thorne, James W. Graham, Samuel A. Agnew, William A. MacKenzie, Leavitt R. Smith, and David A. Heffernan.

A paper "Inspection and Maintenance of Gate Valves," was read by Charles W. Mowry, Manager Inspection Department, Associated Factory Mutual Fire Insurance Companies, Boston, Mass.

A paper "Maintenance of Gate Valves in Boston," was read by George H. Finneran, Superintendent of Water Service, Boston, Mass.

These two papers were discussed by Messrs. Samuel A. Agnew, David A. Heffernan, J. B. Connors, Horace J. Cook, and Brenton W. Vaughan.

(*Adjourned.*)

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*Of Metcalf & Eddy, Engineers, Boston, Mass.

Water Works School.

The second short course in water works management was given during the week of January 23, 1933, by the New England Water Works Association with the coöperation of the staffs of the Departments of Sanitary Engineering of the Harvard Engineering School and of the Massachusetts Institute of Technology, assisted by other specialists.

No one but water works employees was admitted to the course. Non-members of the Association were admitted if application on their behalf was made by a water commission, department or company, or a State Department of Health.

The tuition fee for the course was $15, payable to the Association in advance. The Association suggested that water commissions, departments and companies pay the fees of employees who applied for admission.

The program of the School follows:

Registration, Monday, January 23, 1933, 9:00 to 9:30 A.M.
Room 1041, Massachusetts Institute of Technology.

9:30 A.M. Introduction by Robert Spurr Weston, Chairman Committee on Educational Service, N.E.W.W.A.

Option 1

Design and Operation of Water Works Systems

Monday, January 23

10 A.M. Water Supply Needs — Population, Water Consumption, Fire Flow — Professor Camp, *M.I.T.*

2 P.M. Hydrology — Rainfall, Stream Flow, Evaporation, Floods, Ground Water — Professor Fair, *Harvard*

Tuesday, January 24

10 A.M. Hydraulics — Measurement of Water by Orifices, Venturi Meters, Nozzles, Weirs, Current Meters, Floats and Domestic Meters — Professor Russell, *M.I.T.*

2 P.M. Hydraulics — Conveyance of Water in Pipes and Channels — Professor Russell, *M.I.T.*

Wednesday, January 25

10 A.M. Pumping of Water — Pumps and Pumping Stations — Professor Berry, *Harvard*

2 P.M. Design of Distribution Works — Service Reservoirs, Standpipes and Elevated Tanks, Distribution Systems — Professor Camp, *M.I.T.*

Thursday, January 26

10 A.M. Design of Collection Works — Intakes, Reservoirs, Dams, Pipe Lines and Aqueducts, Wells and Infiltration Galleries — Professor Fair, *Harvard*

2 P.M. Pump Testing — Mr. Smith, *Harvard*

Friday, January 27

10 A.M. Water Works Management — Mr. Caleb Mills Saville, Manager and Chief Engineer, Water Bureau, Metropolitan District Commission Hartford, Conn.

2 P.M. Water Works Accounting — Professor Fiske, *M.I.T.*

Saturday, January 28

10 A.M. Inspection of Service Department of Boston Water Works

OPTION 2

Water Purification and Control of Water Quality

Monday, January 23

10 A.M. Principles of Sanitary Science — PROFESSOR FAIR, *Harvard*

2 P.M. Chemical Examination of Water — PROFESSOR WHIPPLE, *Harvard*

Tuesday, January 24

10 A.M. Bacteriological Examination of Water — MR. WELLS, *Harvard*

2 P.M. Chemical Examination of Water — PROFESSOR WHIPPLE, *Harvard*

Wednesday, January 25

10 A.M. Microscopical Examination of Water — PROFESSOR WHIPPLE, *Harvard*

2 P.M. Natural Purification and Control of Reservoirs — PROFESSOR FAIR, *Harvard*

Thursday, January 26

10 A.M. Chemical Treatment of Water — Coagulation, Aëration, Deferrization, Water Softening — PROFESSOR WHIPPLE, *Harvard*

2 P.M. Corrosion and its Mitigation, Slow Sand Filtration — PROFESSOR FAIR, *Harvard*

Friday, January 27

10 A.M. Laboratory Control of Water Quality — PROFESSOR WHIPPLE, *Harvard*

2 P.M. Rapid Sand Filtration — PROFESSOR CAMP, *M.I.T.*

Saturday, January 28

Inspection of Providence, R. I. Water Purification Works

The School closed with a dinner at the Faculty Club of Harvard University. Mr. Weston presided. The President of the Association, Mr. Ellis, presented the certificates of attendance and Mr. Arthur D. Weston spoke on the Water Supplies of Massachusetts.

Certificates of attendance were given to the following men who attended two sessions of the course under either option each day for five days.

Allen, Earle C., Brockton, Mass.; Brigham, Harold L., Marlborough, Mass.; Calderwood, Donald C., Nashua, N. H.; Cobb, Edwin B., Norwood, Mass.; Crocker, Harold S., Brockton, Mass.; Croker, Mark F., Newton Upper Falls, Mass.; Gassett, Merton A., Brockton, Mass.; Halpin, Henry E., Boston, Mass.; Hancock, Robert F., Lawrence, Mass.; Harrington, Arthur V., Stoneham, Mass.; Herlihy, Michael J., Newtonville, Mass.; Hughes, Edward F., Watertown, Mass.; Libby, Maurice A., Bath, Me.; Mackay, John D., Coscob, Conn.; Melley, William P., East Milton, Mass.; Miller, Charles E., Dedham, Mass.; Morgan, Edward F., Jr., Hudson, Mass.; Newkirk, Samuel F., Jr., Elizabeth, N. J.; Ouderkirk, Lester H., Brockton, Mass.; Robinson, John H., Concord, Mass.; Startard, Lloyd W., Holden, Mass.; Toole, Harold J., Clinton, Mass.

JOHN RIPLEY FREEMAN.

John Ripley Freeman was born in West Bridgton, Maine, July 27, 1855, and died in Providence, R. I., October 6, 1932.

He received his early education in the public schools and graduated from the Massachusetts Institute of Technology, Department of Civil Engineering, in 1876. Prior to his graduation he was employed during summer vacations by the Essex Company, the water power company in Lawrence, Mass., and on graduation was given regular employment. He soon became principal assistant engineer to Hiram F. Mills, Agent of the Essex Company, and assisted Mr. Mills in his many problems in hydraulics and mill construction. During this period he was also inspired by other great engineers with whom he came in contact. Here is where the foundation was laid for his later advance in the fields of hydraulics, mill construction, and business.

In 1886 he was requested by the Associated Factory Mutual Fire Insurance Companies in Boston to join that organization. Against the advice of some of his friends, he accepted this offer which was a turning point in his career. During his connection as special inspector and engineer for these companies, he conducted experiments on the discharge of fire streams, establishing the fact that the nozzle was a good water meter and preparing tables of discharge which won for him the Norman Gold Medal in 1889. He soon discovered that many fire pumps were not adequate for the severe service to which they were put, and, in connection with the builders, prepared standard specifications for the pumps called the "Underwriter" pattern. He also carried on experiments to find the proper relation of pipe sizes to the service to be rendered in connection with automatic sprinkler systems, and the friction losses in ordinary pipe. During this period he reorganized the engineering and inspection departments of the insurance companies.

After ten years in this general inspection and engineering work, he was called to Providence, R. I., as president of three of the Factory Mutual Companies and later was made president of three other companies, which offices he held until his death. The great increase in amount of insurance placed and the great reduction in losses by fire in industrial plants insured in his companies were in part due to his work.

During these years and later, he had an agreement that he could devote one half his time to outside engineering work. One of his earlier outside undertakings of importance was as engineer-member of the Metropolitan Water Board in connection with the water supply for Boston and vicinity from the Wachusett reservoir. Another was in connection with the development of the Charles River basin. Later came his reports on the water

supply of New York, Baltimore, Los Angeles, and San Francisco. From 1905 until his death he was Consulting Engineer to the Board of Water Supply of the City of New York.

Mr. Freeman was appointed three times by the President of the United States a member of engineering boards reporting on special problems of the Panama Canal, and in 1918, a member of the National Advisory Committee for Aeronautics. He also served as a member of the Advisory Committee of the Fire Prevention Section of the War Industries Board. From 1917 to 1920 he acted as consulting engineer to the Chinese Government on improvement of the Grand Canal, including preliminary studies of flood prevention on the Yellow and Hwai Rivers. Later he spent two years of exhaustive study for the Sanitary District of Chicago, on the Great Lakes lowering controversy, in the preparation of a program for controlling the elevation and discharge of Lakes Michigan, Huron, and Erie.

He made several trips to Europe to study the hydraulic laboratories there and persuaded the directors of these laboratories to publish the results of their work. He later arranged for a translation of these articles and, with additional matter, published his book on Hydraulic Laboratory Practice. In his desire to have this country benefit from the work of the hydraulic laboratories in Europe, he established three funds with the American Society of Civil Engineers, the American Society of Mechanical Engineers, and the Boston Society of Civil Engineers, the income from which was to be used to send young engineers, instructors or graduate students for study in hydraulic laboratories in Europe, and for publishing important papers and books on hydraulics.

He conceived the establishment of a national hydraulic laboratory which eventuated in the hydraulic laboratory in connection with the Bureau of Standards at Washington, D. C. He regretted, however, that the laboratory built was not on as large a scale as he believed desirable.

Mr. Freeman's interest in building design caused him to make a study of the causes and effects of earthquakes on structures. He proposed a program of research to obtain better data on earthquake motion for use in design under earthquake stress. His recently published book on "Earthquake Damage and Earthquake Insurance" is probably the most complete compilation of earthquake experience.

He was intensely interested in educational matters, particularly those related to Science and Engineering, and was for forty years a member of the Corporation of the Massachusetts Institute of Technology. To this institution he gave much of his time and money. He was severe in his criticism and helpful in suggestions for improvement of the courses and the personnel of the Institute.

For his great devotion to the engineering profession, Mr. Freeman was honored by election as President of the Boston Society of Civil Engineers, the American Society of Civil Engineers, and the American Society of Mechanical Engineers, and to honorary membership in engineering societies

in this country and abroad. In recognition of his educational work, he has received honorary degrees from several colleges and universities.

His achievements in engineering, business and educational work, and his services to his community and country are too numerous to be given here. They were broad and numerous, covering assignments in this and other countries. His publications in the form of papers, reports, and books on engineering projects and other subjects, disclose his great intellect, breadth of interest, and thoroughness.

Mr. Freeman's outstanding habit of mind was never to take anything for granted, but, as consulting engineer, to go further into the problem than those who were consulting him. Much of his work was of great importance to the New England Water Works Association.

CHARLES T. MAIN.
EDWARD V. FRENCH.
ARTHUR T. SAFFORD.

INDEX OF ADVERTISERS.

	Page
Atlas Mineral Products Co.	xxvi
Barbour, Frank A.	ii
Barrows, H. K.	ii
Bingham & Taylor	v
Blair & Marchant	iii
Builders Iron Foundry	ix
Caldwell, Geo. A., Co.	xv
Cement Lined Pipe Co.	xxvii
Conard, William R.	ii
Crosby, Irving B.	ii
Donaldson Iron Co.	xxvi
Eddy Valve Co.	xvii
Edson Manufacturing Co.	x
Ellsworth, Samuel M.	iii
Fairbanks-Morse & Co.	xxi
Fay, Spofford & Thorndike	ii
Ferullo, G., Co.	iii
Ford Meter Box Co.	xix
Fuller & Everett	ii
Fuller & McClintock	ii
Gamon Meter Co.	xiii
General Pipe Cleaning Co.	xix
Goodnough, X. Henry, Inc.	ii
Golden-Anderson Valve Specialty Co.	xxix
Hayes Pump and Machinery Co.	xxxiv
Hays Mfg. Co.	xx
Hersey Mfg. Co.	iv
Hughes, Edward F.	iii
Hydraulic Development Co.	xxiv
Kennedy Valve Mfg. Co.	xvi
Lead Lined Iron Pipe Co.	xxvi
Leadite Co.	Back cover
Lock Joint Pipe Co.	Facing front cover
Ludlow Valve Mfg. Co.	viii
Main, Charles T., Inc.	iii
Main, M. B.	xxv
Massur, F. A., & Co.	xix
Metcalf & Eddy	ii
National-Boston Lead Co.	xviii
National Meter Co.	xiv
National Water Main Cleaning Co.	xxxiv
Neptune Meter Co.	vi
New England Water Works Association	xxvii
Pirnie, Malcolm	iii
Pierce-Perry Co.	xxvii
Pitometer Company	iii
Pittsburgh Equitable Meter Co.	vii
Rensselaer Valve Co.	xxv
Ross, Anthony & Son	iii
Ross Valve Mfg. Co.	xxxiv
Simplex Valve & Meter Co.	xii
Smith, The A. P., Mfg. Co.	xix
Sumner & Dunbar	xxvii
Thorpe, Lewis D.	iii
Turbine Equipment Co.	xxii
Union Water Meter Co.	xi
U. S. Cast Iron Pipe and Foundry Co.	Facing back cover
Wallace & Tiernan	xxiii
Warren Foundry and Pipe Co.	x
Weston & Sampson	ii
Whitman & Howard	ii
Wood, R. D., & Co.	xxviii

(Classified index on page xxxii)

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The "blue chip" stock which pays regular dividends
or
The quality water meter —reasonably priced— which produces just revenue at low cost.
THE POOR INVESTMENT
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The "speculative" stock which pays no dividends and is good only for papering the walls
or
The water meter whose sole qualification is cheap initial price.

Quality—*not Price* distinguishes the good WATER METER INVESTMENT

Water departments all over the country know that Hersey Water Meters are a good investment. The high standards of their material, workmanship and design insure regular dividends at low maintenance cost for many years to come.

No matter how small the budget may be, it is a mistake to put your money into anything but a quality product. A note to the nearest Hersey branch office will bring you complete information about the quality advantages of the Hersey Meter.

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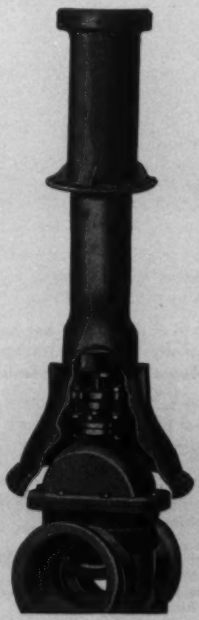
The cuts shown are our two and three piece sliding type adjustable valve boxes.

The flange on the bottom of the top section can be located any distance from the top.

We carry a complete stock on hand at all times.

Wire, phone or write us when you are in a hurry for a shipment.



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"PIONEERS in Meter Progress" a phrase, a slogan, yet far more important than both.

For more than a generation TRIDENT & LAMBERT Meters have pioneered, but not simply in mechanical meter improvements.

For years we have been in the very forefront of the battle to reduce water wastes and to increase the revenue and cut the costs in Water Departments all over the country.

Time and again it has been proved that TRIDENT and LAMBERT Meters have signally reduced wastes and have enabled municipalities to show greater income and less operating costs.

Never was this so vitally important as today. Never before have TRIDENT and LAMBERT Meters offered such opportunities to show greater returns from the sale of water, without increasing the charge for water used.

And, back of TRIDENT and LAMBERT performance, is a history of pioneering in meter improvements unmatched in this field. Among outstanding TRIDENT achievements are:

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The First Fire Service Meter officially approved by the Underwriters' Laboratories, Inc., TRIDENT PROTECTUS.

Write for catalogs to the Neptune Meter Co. (Thomson Meter Corp.), 50 East 42nd Street, New York City . . . or Neptune Meter Co., Ltd., Toronto, Ont.

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"Cash Registers of the Water Works Field"

For Remote Indicating

Telemco System

TYPICAL TELEMCO INSTALLATION

Water level in reservoir indicated at pumping station

. . A MEANS OF REMOTE INDICATION of any measurement, such as water level in reservoirs, penstocks, streams, pressure and flow in distribution systems, from any location—by private wire or leased telephone line—over any distance—to any point.

. . ACCURATE INFORMATION at a central location at all times without costly manual operation.

. . BETTER PUBLIC RELATIONS by keeping best possible supply and pressure conditions throughout distribution systems.

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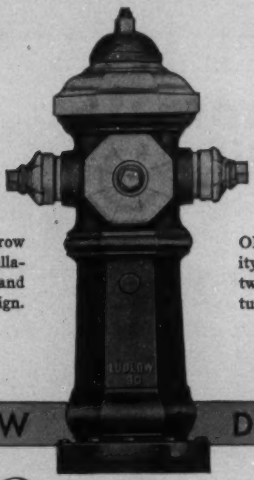

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Specify Ludlow Double Gate Valves. Available in Iron, Bronze or Steel up to 78 inches. Prompt deliveries from stock. 8,000,000 Ludlow Valves and Hydrants now in use.

The Ludlow Diamond Fire Hydrant is less than two years old. Already it has been adopted in over 100 cities in 26 different states.

Residential building has been at such a standstill the last two years that water works extension has been reduced to an absolute minimum. Hence for so many cities—scattered all over the country—to "go Ludlow" is proof that the Ludlow Diamond is more than just a new Hydrant. It has definite points of superiority which any city—your city—will quickly recognize. Why not investigate this matter now?

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Fire Hydrants—Double and Single Gate Valves — Air, Check and Foot Valves—Sluice Gates—Power Operated Valves

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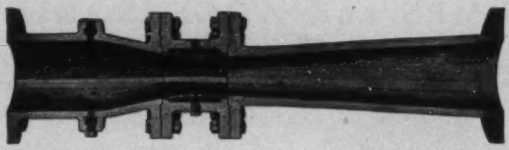

The Venturi Tube

In 1891 the Venturi Tube was new. Those Engineers who knew of it hoped it would help solve the difficult problem of measuring the flow of water in large pipe lines.

In 1933 the Venturi Tube is accepted by Engineers as the most dependable and accurate means of measuring flow through pipe lines.

It may be worthwhile, however, to summarize a few of the important advantages of the Venturi Tube:

1. The flow is guided at all points.
2. The inlet and throat pressures are taken off at definitely fixed points; the annular pressure chambers assure the use of true average pressures.
3. The Venturi throat is accurately centered in relation to the rest of the pipe line.
4. The friction loss through a Venturi Tube is the lowest of any differential producer. The Venturi Meter, then, operates on a greater differential with less friction loss, and is a more powerful instrument with a wider measuring range than other meters.
5. The coefficients of the Venturi Tube have been definitely established in numerous calibration tests of tubes ranging in size from 1 inch to 8 feet in diameter.

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"Builders of the Venturi since 1891"

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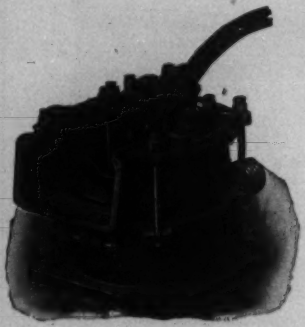

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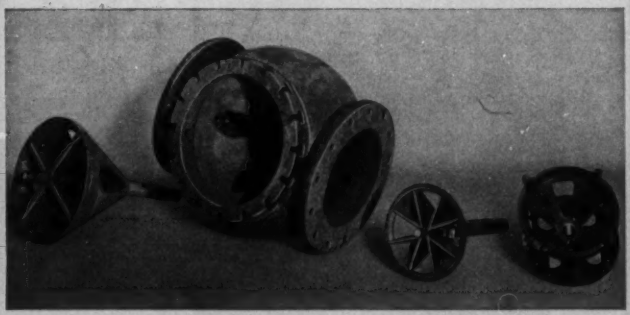


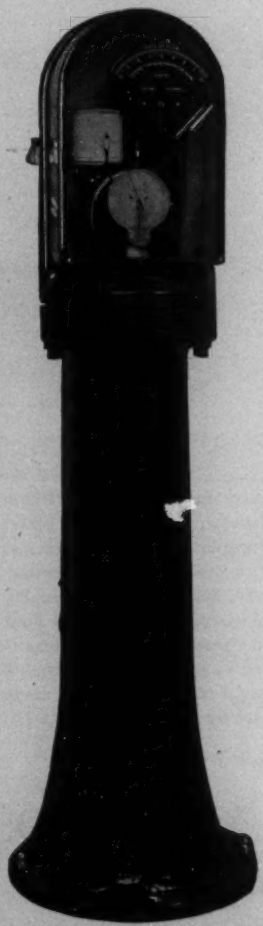
THE OLD AND THE NEW

In 1866, Union Water Meter Company furnished a 12″ Union Water Pressure Regulator for the Camden and Rockland Water Company. For two generations, this regulator was in continuous service. Recently it came back home to the factory for reseating. Union Water Meter Company has always made the highest quality meters and allied products; long and continuous service is built into them. But our specialists have not been unmindful of opportunities for improvement as is indicated by the present-day type of 6″ Union Pressure Regulator on the right of the photograph made with removable seat, and with more portage capacity than its 12″ grandfather.

UNION WATER METER CO.

WORCESTER, MASS., U. S. A.

WATER METERS
CHRONOMETER VALVES
EASY-TURNING STOPS
PRESSURE REGULATORS



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VENTURI TYPE
METERS

Indicate the flow on a uniformly graduated dial.

Record the flow on a rectangular chart with uniform graduations.

Totalize the flow on a five-dial totalizer, so designed that the last figure can be read with accuracy on a much larger scale.

They may be furnished for floor stand, wall bracket or panel mounting.

May be used for pump discharge—gravity supply—in the Filter Plant—or Sewage Treatment Works.

Simplex Meter Registers may be checked at any time while in service by use of apparatus furnished as standard equipment—an exclusive feature of the Simplex Meter.

Simplex Meters are the choice of many who value quality and accuracy above initial cost.

Write for data.

Simplex Valve and Meter Co.

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"WATCH DOG" WATER METERS

ACCURATE *DURABLE*

DISC, CURRENT AND COMPOUND TYPES

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METERS

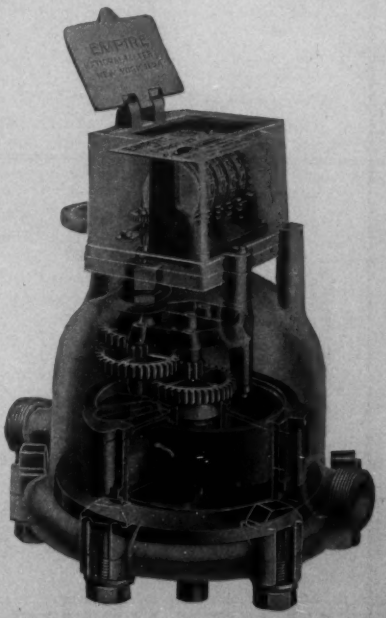
That Turn Your Trickles into Cash

There's no magic about it either. EMPIRE meters will register little leaks and streams that wouldn't even move an ordinary meter, and enable you to collect real money for them.

The Oscillating Piston Does it

They are money savers too, in more ways than one. A special point is, that EMPIRE cylinders do not have to be replaced when they show a little wear. Original accuracy can be restored by re-machining at trifling cost, and the meter is all ready for another long stretch of hard service.

Think of that when figuring on upkeep.



All Sizes
⅝ to 6 Inches

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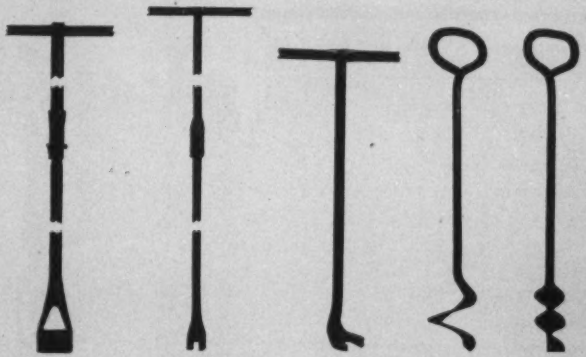

A-106 A-107 A-108 A-109 A-110

A-106. Sectional Gate Valve Key. Each Section 30″ long. Made of Highest Grade Seamless Drawn Steel Tubing.

A-107. Sectional Curb Cock Key. Each Section 30″ long. Made of same material as Valve Key.

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Our Gate and Curb Box Keys are the Lightest and Strongest on the market. We can furnish these in any length without the Sectional feature.

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A-104. "Caldwell" Plugging Chisels for removing Calking Material from C.I.P. Joints. Made in two sizes.

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The city chiefs wanted PROOF

—so we crashed into the hydrant and repaired it within ½ hour

The Kennedy SAFETOP Fire Hydrant

Send for full information

ON December 16, a number of water department superintendents and engineers of the State of Maine watched while an automobile truck smashed into a Kennedy SAFETOP Fire Hydrant which was connected to the supply main with full city water pressure of 80 lbs.

When the heavy truck struck the SAFETOP Hydrant, the patented Safety Breakable Section broke cleanly apart. The entire upper standpipe section assembly, dislodged from the base, fell to the ground. The driver was merely jarred, and the only damage to the truck was a bent bumper. No part of the upper or lower hydrant standpipe or stem sections or operating mechanism was damaged, nor was there any flood.

The superintendents and engineers were amazed, but an additional surprise awaited them. After the truck was backed away, the Kennedy representative started to repair the broken hydrant. It was a bitterly cold day, and the sub-zero temperature made it difficult to handle the castings, but in spite of this handicap, a new standpipe breaking ring and stem coupling were set in place, the upper standpipe section assembly replaced, and the hydrant was again ready for service in 26 minutes—the repair parts costing $6.00.

No excavating was required, nor was it necessary to shut off the water pressure at any time, and when the hydrant was opened at the conclusion of the test, it was found to be in perfect operating condition.

The Kennedy Valve Mfg. Co.
ELMIRA, N. Y.

KENNEDY

VALVES~PIPE FITTINGS~FIRE HYDRANTS

More Than Sixty Years of Experience Goes Into All

EDDY

Hydrants and Valves

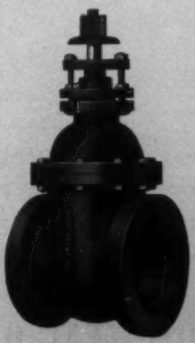
BUILT TO MEET EVERY EXACTING REQUIREMENT OF WATER-WORKS SERVICE

Type of Eddy Hydrant Popularly Used

Practical water-works men instantly recognize the superior features of Eddy products. For instance, with the hydrant.

A damaged valve can be replaced in a few minutes — Water hammer is avoided by the valve closing against the pressure and the use of the cut-off under the valve, which gradually closes it —
Stems cannot be bent in closing the hydrant.
Nothing can lodge on the valve seat to prevent closing of the valve —
The Drip Rod can be removed without interfering with the hydrant in any manner —
The position of the stem of the hydrant indicates accurately the position of the valve —
Whatever can pass the valve opening will pass the standpipe —
No frost case necessary —
All of these points are explained thoroughly in our catalogue, which will pay you to study.

The Eddy Valve, Too



Has its distinctive and individual features. The double disk gates are free to adjust themselves in different positions every time the valve is closed, thereby keeping smooth and tight faces. And two bronze hooks on the ball loosely engage with the gates to prevent them from spreading at the top. The center bearing gates are forced to their seats with equal pressure at all points. There is a good deal more worth knowing about Eddy Valves. Further details will be sent for the asking.

Other Eddy Products

The Eddy line of water-works products also includes horizontal and vertical check valves, foot valves, hydraulically operated valves, indicator posts, etc.
All goods made by the Eddy Valve Company are manufactured exclusively at Waterford, New York.

WRITE FOR CATALOGUE

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WATERFORD, N. Y., U. S. A.

National-Boston Lead Co.

800 Albany St., Boston

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(Every Atom Pure Lead)

For calking pipe joints under the most difficult conditions.

For overhead joints, or in wet places where the use of molten lead is not only impracticable but dangerous — **LEAD WOOL** may be used to advantage.

It makes an absolutely tight joint which will withstand the highest pressure, yet be sufficiently elastic to allow considerable sagging or settling of the pipe without danger of a leak.

As compared with the poured joint, in this respect, the superiority of **LEAD WOOL** is apparent.

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having high coefficients deliver more water and give better fire protection than corroded mains.

High coefficients mean greater delivery in gravity mains and lower power costs in pumping mains.

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—We would be glad to send a folder, on your request, describing how we recently made this difficult cut in an hour, using a—

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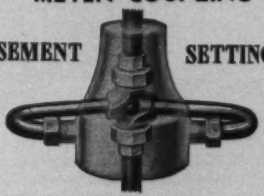
THE A.P. SMITH MFG. CO. East Orange, N.J.

The

COPPERHORN

METER COUPLING

for BASEMENT SETTINGS



- ●1 **Meter set horizontally** at desired height in vertical pipe without ells, nipples or couplings.
- ●2 **All copper water way.** No rusting or corrosion. Freezing will not burst it.
- ●3 **Smooth oversize tubing** and sweeping bends, maximum flow.
- ●4 **Meter changes** do not disturb piping whether the meter is in or out.
- ●5 **On new services** it serves as a spacer to connect in the piping.
- ●6 **Provides a permanent** electrical ground connection.

May We Give You Further Information About the Copperhorn?

The FORD METER BOX COMPANY

Wabash, Indiana

The Hays Copper Plumbing Method

Regular Corporation Stop

Adapter

In making an installation with Hays Copper Service Fittings the regular Corporation Stops can be used in conjunction with the adapter. It is not necessary to procure new mandrels, as the Corporation Stops are inserted with the regular mandrels, in the usual manner, after which the adapter is attached.

Open View of Adapter

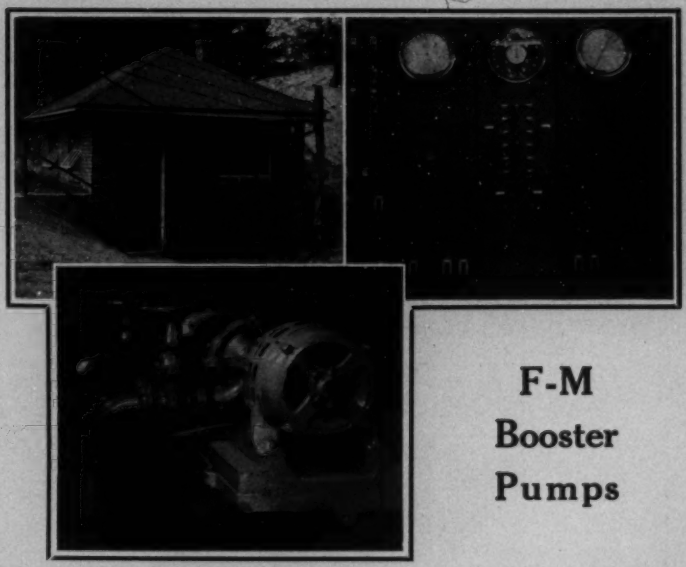
These adapters are made with Female Thread for Increased I. P., Water Works Standard or Lead Flange.

To use standard Corporation Stops with Increased I. P. or Water Works Standard Thread, and where a direct connection of the Corporation Stop and Copper Pipe is desired, the above stop has been designed. (See cut above for sectional view of joint.) These stops permit a Hays 90° seal with the use of regular male mandrels.

Write for complete details of

The Hays Copper Plumbing Method for Water Works and House Plumbing

Hays Manufacturing Co. Erie, Pa.



F-M
Booster
Pumps

At SOUTH HADLEY, MASS.

Shown above are the exterior of the South Hadley Water Works Pumping Station, the two F-M Motor Driven Centrifugal Pumps installed in the station and the control equipment. The station is located at one extreme end of the town and takes water from a small reservoir under a positive head of approximately 12 or 15 feet and pumps it through the main section of the town to a standpipe at the other end. Both pumps and motors are of ball bearing design, which construction was selected because of the desirable feature of lubrication. The bearings require attention only once a year.

FAIRBANKS, MORSE & CO.

178 Atlantic Avenue, Boston, Mass.

FAIRBANKS-MORSE

PUMPS · DIESEL ENGINES · MOTORS

DE LAVAL

First of Three De Laval Geared Turbine Driven Pumps and Generators at Needham Street Station

NEWTON WATER WORKS

Photograph Taken Five Weeks After Arrival at Station

Installed by

TURBINE EQUIPMENT COMPANY

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Selected as a camp site by Gaspar de Portola, August 1st, 1769, because of the available fresh water flowing down from the nearby San Gabriel foothills (named by Father Juan Crespi, the following day—the day of "Our Lady, Queen of the Angels"), Los Angeles owes its very existence to PURE WATER.

On the public and private swimming pools of Los Angeles, on sewage disposal plants and allied sanitary installations, 42 more W&T chlorinators are giving this dependable service in Los Angeles today.

LOS ANGELES *owes its existence to* PURE WATER

THE supply of water from the San Gabriel foothills is no longer adequate and Los Angeles spent millions to tap an available supply 260 miles distant.

And to maintain the purity of this water, all 14 divisions of the Los Angeles supply are today chlorinated entirely by W&T Vacuum Chlorinators—two-thirds of them fully automatic in operation.

On 22 occasions since 1923 Los Angeles has purchased W&T equipment—36 Vacuum Chlorinators in daily service give that same dependability, that same reliable service demonstrated three years ago when the St. Francis Dam went out. Of this, Dr. Carl Wilson, Director of Sanitation, says in "Western Water Works": "It is interesting to know that the Lower San Fernandino plant on the Los Angeles Municipal Water System actually performed as planned by its designers when unprecedented floods created a condition of emergency". W&T Vacuum Automatic Chlorinators *automatically* and *adequately* chlorinated the greatly increased flow to avert the disaster of water borne disease.

Dependable chlorination can be yours, too, with the installation of W&T Vacuum Chlorinators.

Ask for your copy of Technical Publication 38

WALLACE & TIERNAN CO., INC.

Manufacturers of Chlorine and Ammonia Control Apparatus

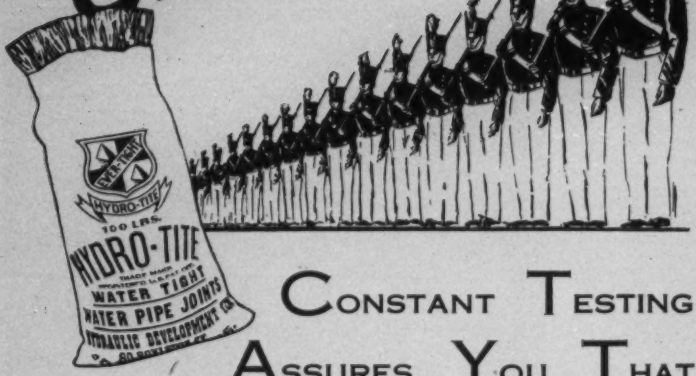
NEWARK, NEW JERSEY Main Factory: BELLEVILLE, NEW JERSEY

Branches in Principal Cities

"The Only Safe Water Is a Sterilized Water"

SA-42

UNIFORMITY



CONSTANT TESTING ASSURES YOU THAT

WHEN you receive a bag of Hydro-Tite you receive 100 lbs. of material which is exactly like the last Hydro-Tite you used. Every manufacturing operation is carefully inspected and many of them checked by actual test. This care in compounding is only possible when a powdered material is made as we make Hydro-Tite.

Hydro-Tite joints are tight, strong and flexible. They are made without caulking — have a record of over 20 years and save from 50% to 75% as compared with lead. Write for information on any phase of joint making.

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HYDRO-TITE

A DEPENDABLE SELF-CAULKING JOINT COMPOUND

Main's Hotcoating for the inside of standpipes and water tanks, will stop all pitting and corroding, and is guaranteed for seven years.

Also

Main's Coverall Salt Air Paint, for the outside of standpipes and water tanks, guaranteed for four years.

M. B. MAIN **Box 189, Somers Point, N. J.**

The "COREY" Hydrant

Why you should use GENUINE COREY HYDRANTS

Water to Nozzles quicker than other Hydrants.

Standpipe can be removed and replaced without shutting off water in main.

Ease of operation insured by knuckle joint principle.

Only 13 turns to completely open or completely close, but full flow with only 4 turns.

Nozzles screwed in — not leaded.

Can be lengthened without shutting off water.

Will not flood if broken.

Positive Drip — cannot freeze.

Standpipe can be revolved quarter turns — Nozzles always in right direction.

Ask for our Catalog "G"

CHARLES L. BROWN,

New England Sales Manager, Lock Box 2, Northboro, Mass.

RENSSELAER VALVE COMPANY

TROY, N. Y.

DONALDSON IRON CO.

MANUFACTURERS
...OF...

CAST IRON PIPE

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Special Castings for Water and Gas.
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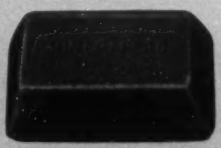

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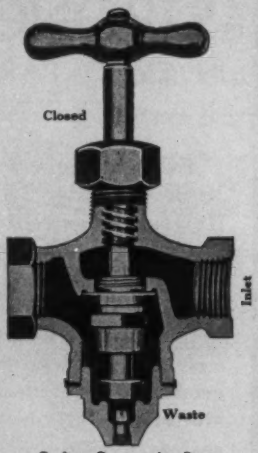

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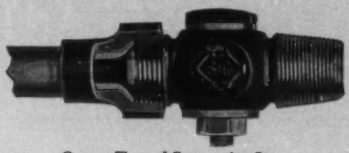

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CLASSIFIED DIRECTORY OF ADVERTISEMENTS.

AIR COMPRESSORS. PAGE
Hayes Pump and Machinery Co. . . . xxxvi

BOILERS, STEAM.
Massur, F. A., & Co. . . . xix

BRASS GOODS.
(See also Pipe, Brass.)
Caldwell, George A., Co. . . . xv
Cement Lined Pipe Co. . . . xxvii
Hays Mfg. Co. . . . xx
Pierce-Perry Co. . . . xxvii
Sumner & Dunbar . . . xxvii
The A. P. Smith Mfg. Co. . . . xix
Union Water Meter Co. . . . xi

CALKING MACHINERY AND TOOLS.
The A. P. Smith Mfg. Co. . . . xix

CAST IRON PIPE. (See Pipe, Cast Iron.)

CEMENT LINED PIPE. (See Pipe, Cement Lined.)

CHEMICAL FEED APPARATUS.
Wallace and Tiernan Co., Inc. . . . xxiii

CHLORINATORS.
Wallace & Tiernan Co., Inc. . . . xxiii

CLEANING WATER MAINS.
General Pipe Cleaning Co. . . . xix
National Water Main Cleaning Co. . . . xxxiv

COCKS, CURB, AND CORPORATION.
Caldwell, George A., Co. . . . xv
Hays Mfg. Co. . . . xx
Pierce-Perry Co. . . . xxvii
Sumner & Dunbar . . . xxvii
The A. P. Smith Mfg. Co. . . . xix
Union Water Meter Co. . . . xi

CONCRETE PIPE.
(See Pipe, Concrete.)

CONDENSERS.
Hayes Pump and Machinery Co. . . . xxxiv
Massur, F. A., & Co. . . . xix

CONSTRUCTION EQUIPMENT.
Edson Mfg. Corp. . . . x

CONTRACTORS.
Ferullo, G., Co. . . . iii
Hughes, Edward F. . . . iii
National Water Main Cleaning Co. . . . xxxiv

CURB BOXES.
Bingham and Taylor . . . v
Caldwell, George A., & Co. . . . xv
Hays Mfg. Co. . . . xx
Sumner & Dunbar . . . xxvii

CURB AND VALVE BOX FINDERS.
Caldwell, George A., Co. . . . xv

DIAPHRAGMS, PUMP.
Edson Mfg. Corp. . . . x

ENGINEERS.
Barbour, Frank A. . . . ii
Barrows, H. K. . . . ii
Blair & Marchant . . . iii
Fay, Spofford & Thorndike . . . ii
Fuller & McClintock . . . ii

CLASSIFIED DIRECTORY OF ADVERTISEMENTS *(Continued)*.

ENGINEERS — *Continued* — PAGE

Ellsworth, Samuel M . . . iii
Fuller & Everett . . . ii
Goodnough, Inc., X. Henry . . . ii
Main, Charles T. . . . iii
Metcalf & Eddy . . . ii
Pirnie, Malcolm . . . iii
Ross, Anthony & Son . . . iii
Thorpe, Lewis D. . . . iii
Weston & Sampson . . . ii
Whitman & Howard . . . ii

ENGINES.
(See Pumps and Pumping Engines.)

ERECTORS, WATER WORKS AND POWER MACHINERY.
Hayes Pump and Machinery Co. . . . xxxiv
Massur, F. A., & Co. . . . xix
Turbine Equipment Co. . . . xxii

FEED WATER HEATERS.
Hayes Pump and Machinery Co. . . . xxxiv
Massur, F. A., & Co. . . . xix
Ross Valve Mfg. Co. . . . xxxiv

FILTER RATE CONTROLLERS AND GAGES.
(See Rate Controllers.)

FILTERS AND WATER SOFTENING PLANTS.
Ross Valve Mfg. Co. . . . xxxiv

FILTRATION PLANT EQUIPMENT.
Builders Iron Foundry . . . ix
Ross Valve Mfg. Co. . . . xxxiv
Wallace & Tiernan Co., Inc. . . . xxiii

FLEXIBLE JOINTS.
U. S. Cast Iron Pipe and Foundry Co. . . . Facing back cover

FURNACES, ETC.
Atlas Mineral Products Co. . . . xxvi
The A. P. Smith Mfg. Co. . . . xix
The Leadite Co. . . . Back cover

GAGES, SURFACE, RESERVOIR AND SPECIAL WATER WORKS.
Builders Iron Foundry . . . ix

GATE VALVES. (See Valves.)

GEOLOGISTS.
Crosby, Irving B. . . . ii

HOSE, SUCTION AND CONDUCTION.
Edson Mfg. Corp. . . . x

HYDRANTS, FIRE.
Eddy Valve Co. . . . xvii
Kennedy Valve Mfg. Co. "Newtype" . . . xvi
Ludlow Valve Mfg. Co. . . . viii
Rensselaer Valve Co. "Corey" . . . xxv
Ross Valve Mfg. Co. . . . xxxiv
The A. P. Smith Mfg. Co. . . . xix
Wood, R. D., & Co. . . . xxviii

LEAD.
National-Boston Lead Co. . . . xviii

LEAD PIPE.
(See Pipe, Lead.)

LEAD WOOL.
National-Boston Lead Co. . . . xviii

LEAK FINDERS.
Pitometer Co. . . . iii

LIQUID CHLORINE.
(See Chlorine, Liquid.)

METERS, WATER AND OIL.
Builders Iron Foundry . . . ix
Gamon Meter Co. . . . xiii
Hersey Mfg. Co. . . . iv

CLASSIFIED DIRECTORY OF ADVERTISEMENTS (*Continued*)

METERS — *Continued* — PAGE

National Meter Co. . . . xiv
Neptune Meter Co. . . . vi
Pitometer Co. . . . iii
Pittsburgh Equitable Meter Co. . . . vii
Simplex Valve & Meter Co. . . . xii
Union Water Meter Co. . . . xi

METER COUPLINGS.

Ford Meter Box Co. . . . xix

METERS (VENTURI TYPE).

Builders Iron Foundry . . . ix
National Meter Co. . . . xiv
Simplex Valve & Meter Co. . . . xii

METER BOXES.

Bingham and Taylor . . . v
Ford Meter Box Co. . . . xix

METER TESTERS.

Ford Meter Box Co. . . . xix

OIL ENGINES, DIESEL.

Fairbanks-Morse & Co. . . . xxi
Hayes Pump and Machinery Co. . . . xxxiv

PIPE, BRASS.

Pierce-Perry Co. . . . xxvii
Sumner & Dunbar . . . xxvii

PIPE, CAST IRON (AND FITTINGS).

Builders Iron Foundry . . . ix
Donaldson Iron Co. . . . xxvi
U. S. Cast Iron Pipe and Foundry Co. . . . Facing back cover
Warren Foundry and Pipe Co. . . . x
Wood, R. D., & Co. . . . xxviii

PIPE, CEMENT LINED.

Cement Lined Pipe Co. . . . xxvii
Union Water Meter Co. . . . xi

PIPE, CONCRETE.

Lock Joint Pipe Co. . . . Facing front cover

PIPE CUTTING MACHINES.

The A. P. Smith Mfg. Co. . . . xix

PIPE JOINTING MATERIAL.

Atlas Mineral Products Co. . . . xxvi
Hydro-Tite . . . xxiv
The Leadite Co. . . . Back cover

PIPE, LEAD.

National-Boston Lead Co. . . . xviii

PIPE, WROUGHT IRON AND STEEL.

Pierce-Perry Co. . . . xxvii
Sumner & Dunbar . . . xxvii

PITOMETERS.

Pitometer Co. . . . iii

PRESSURE REGULATORS.

Caldwell, George A., Co. . . . xv
Golden-Anderson Valve Specialty Co. . . . xxix
Ross Valve Mfg. Co. . . . xxxiv
Union Water Meter Co. . . . xi

PROVERS, WATER.

Pittsburgh Equitable Meter Co. . . . vii

PUMPS AND PUMPING ENGINES.

Edson Manufacturing Corp. . . . x
Fairbanks-Morse & Co. . . . xxi
Hayes Pump and Machinery Co. . . . xxxiv
Massur, F. A., Co. . . . xix
Ross Valve Mfg. Co. . . . xxx
Turbine Equipment Co. . . . xxxiv
Wood, R. D., & Co. . . . xxviii

CLASSIFIED DIRECTORY OF ADVERTISEMENTS *(Continued).*

RATE CONTROLLERS. Page
Builders Iron Foundry ix
Simplex Valve & Meter Co. xii

SLEEVES AND VALVES, TAPPING.
Ludlow Valve Mfg. Co. viii
Rensselaer Valve Co. xxv
The A. P. Smith Mfg. Co. xix

SPECIFICATIONS, FORM AND METER RATE SHEETS
New England Water Works Association xxvi

SULPHATE OF ALUMINA. (See Alum.)

SUPPLIES AND TOOLS.
Caldwell, George A., Co. xv
Hays Mfg. Co. xx
Pierce-Perry Co. xxvii
Sumner & Dunbar xxvii
The A. P. Smith Mfg. Co. xix
The Leadite Co. . . . . Back cover

TANK PAINT.
M. B. Main xxv

TAPPING MACHINES.
Hays Mfg. Co. xx
The A. P. Smith Mfg. Co. xix

TAPPING SLEEVES. (See Sleeves and Valves, Tapping.)

VALVE BOXES.
Bingham and Taylor v
Ford Meter Box Co. xix
Rensselaer Valve Co. xxv
Wood, R. D., & Co. xxviii

VALVE INSERTING MACHINES.
The A. P. Smith Mfg. Co. xix

VALVE SPECIALTIES.
Golden-Anderson Valve Specialty Co. xxix

VALVES, GATE.
Eddy Valve Co. xvii
Kennedy Valve Mfg. Co. xvi
Ludlow Valve Mfg. Co. viii
Rensselaer Valve Co. xxv
Ross Valve Mfg. Co. xxxiv
Simplex Valve & Meter Co. xii
The A. P. Smith Mfg. Co. xix
Wood, R. D., & Co. xxviii

VALVES, REGULATING.
Golden-Anderson Valve Specialty Co. xxix
Ross Valve Mfg. Co. xxxiv
Union Water Meter Co. xi

WATER WASTE DETECTION.
Pitometer Co. iii
Simplex Valve & Meter Co. xii

WROUGHT IRON PIPE. (See Pipe, Wrought Iron and Steel.)

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